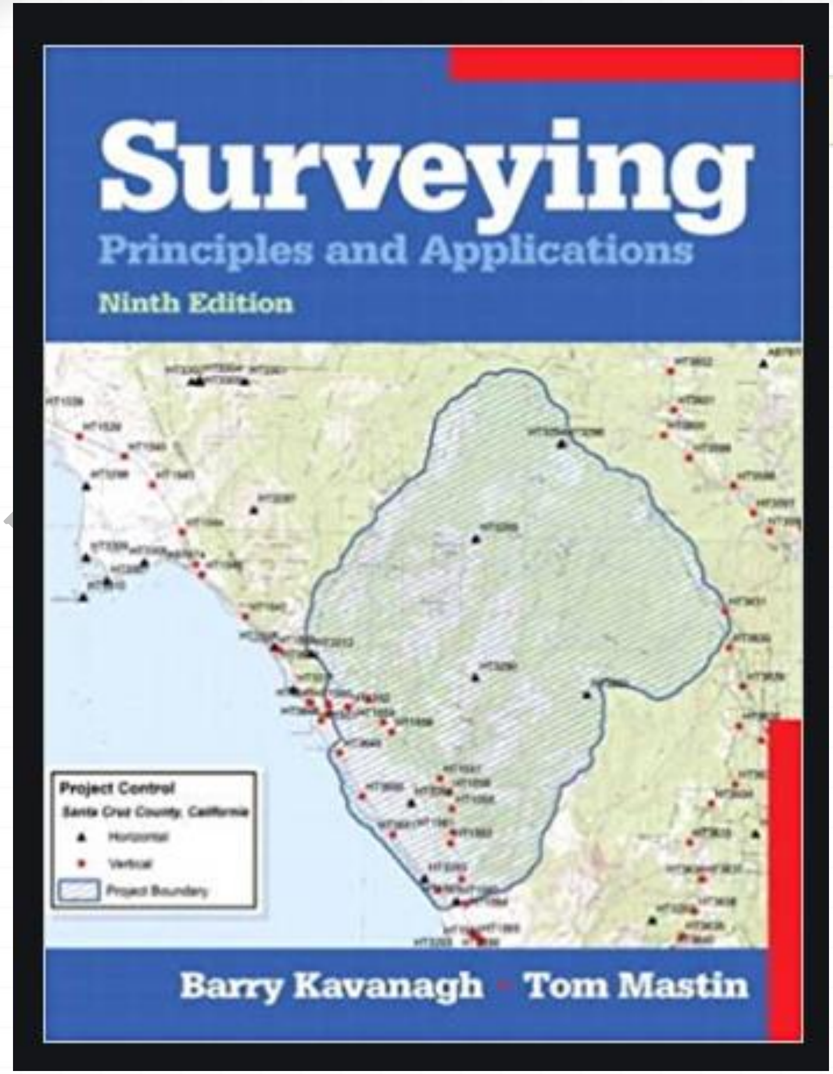
Civilittee
اللجنة الأكاديمية لقسم الهندسة المدنية
www.Civilittee.com

سلايدات

مساحة

د. طالب الروسان


Prof. TALEB AL-ROUSAN
Surveying
2104011365
Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University



Surveying: Principles and Applications
9th Edition
Barry Kavanagh &
Tom Mastin
2014

Prof. TALEB AL-ROUSAN

Chapter 1
Basics of Surveying

What is Surveying?

- The art of making measurements of the relative positions of natural and man-made features on the Earth's surface, and the presentation of this information either graphically or numerically.

- ***The art of measuring distances, angles, and position on or near surface of earth.***

Surveying Branches

Plane surveying: is the process of surveying by assuming that the earth is flat. Which mean the curvature or spherical shape of the earth is not considered in plane surveying calculations.

Geodetic surveying: is a process of surveying by considering the curvature or spherical shape of the earth. The exact positions of points obtained on plane surveying are given by geodetic.

Difference Between Plane and Geodetic Surveying

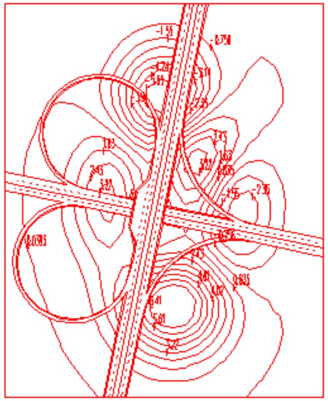
Plane Surveying

- Earth surface is assumed as plain
- Lines formed by any two points are considered as straight line – as the same angles are plain angles
- Suitable for small area surveying.
- ***American survey put 250 km^2 for treating survey as Plane.***
- Survey accuracy is low
- Economic and easy survey method

Geodetic Surveying

- Earth surface is considered as spherical
- Line formed by joining any two points are considered as arch – as the same angles are spherical angles
- Suitable for large area surveying
- Survey accuracy is high
- Special instrument needed and long survey method

Classes of surveys



1- The preliminary survey: (data gathering)

Collection of distances, angles, and difference in elevation data to locate physical features so data can be plotted to scale on a map.

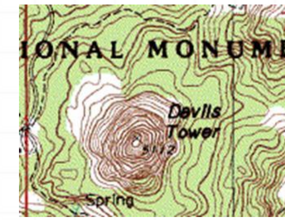
2- Layout surveys: Making on the ground the features shown on a design plan (using wood stakes, iron bars, aluminum and concrete monuments, nails, etc.).

3. Control surveys: To reference preliminary and layout surveys.

- ***Horizontal control can be anything but usually roadways or coordinate control stations.***
- ***Vertical controls are a series of benchmarks having X, Y & Z coordinate.***

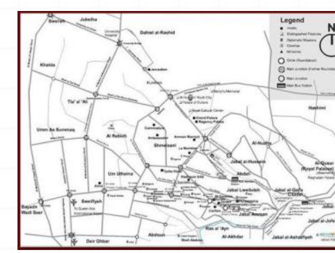
Types of Surveys

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



- **Hydrographic surveys:** Used to tie in underwater features to surface control points. Usually shorelines, marine features, and water depths are shown on the hydrographic map→ Preliminary surveys

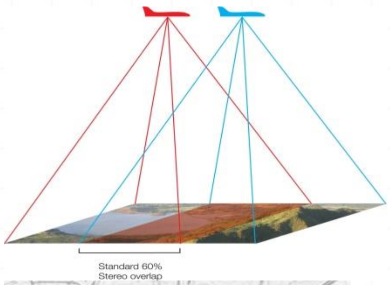
- **Route surveys:** They range over a narrow but long strip of land. Like highways, railroads, …etc. → preliminary+ layout+ control surveys



- **Property surveys (cadastral or land surveys):** Determining boundary locations or laying out new property boundaries → preliminary+ layout+ control surveys

Types of Surveys

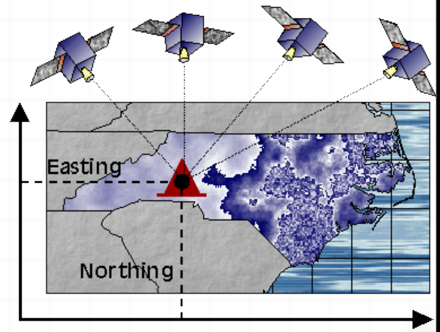
- **Final ("as built") surveys:** similar to preliminary surveys. Final surveys tie in features that have been constructed to provide a final record of the construction and to check that the construction has proceeded according to the design plans.



- **Aerial surveys:** preliminary and final surveys that use both traditional aerial photography and aerial imagery.

- **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)→ layout surveys

- **GPS:** N, E and elevation using NAVSTAR satellite signal.



The Role of Surveying in Civil Engineering Practice

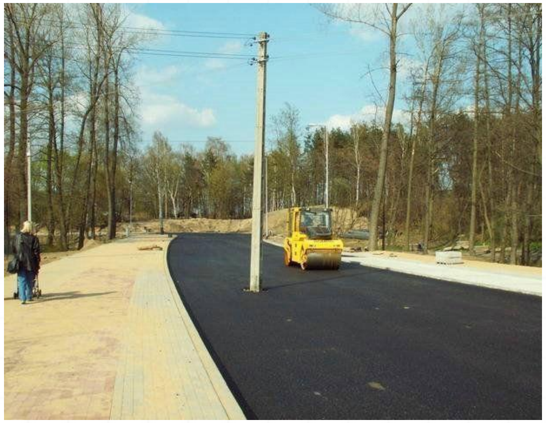
Surveyors are needed:

• to maintain the geometric order during the construction process.

• to provide fundamental data for the design and planning process

• to provide quantity control during the construction process (for example: earthwork quantities)

• to monitor the structure after the construction (subsidence, deformations, etc.)



Surveying Instruments

1. Chain and Tape

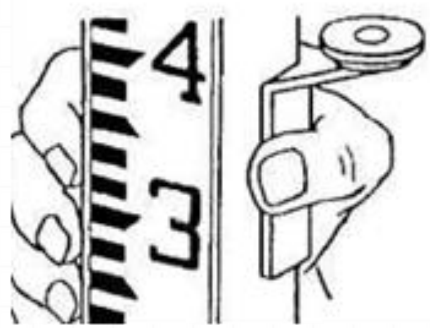
Chains or tapes are used to measure distances on the field.
A **chain** is made up of connected steel segments, or links, which each measure 20 cm. Usually, a chain has a total length of 20 meters (66 ft), including one handle at each end (100 links).

Measuring tapes: are made of steel, coated linen, or synthetic material. They are available in lengths of 20, 30 and 50 m. Centimeters, decimeters and meters are usually indicated on the tape.

Surveying Instruments

2. Measuring Rod (level staff or graduated rod)

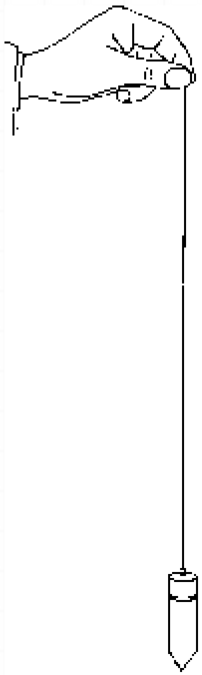
A measuring rod is a straight lath with a length varying from 2 m to 5 m. The rod is usually marked in the same way as a measuring tape, indicating centimeters, decimeters and meters.



Surveying Instruments

3. Plumb Bob

A plumb bob is used to check if objects are vertical. A plumb bob consists of a piece of metal (called a bob) pointing downwards, which is attached to a cord. When the plumb bob is hanging free and not moving, the cord is vertical.

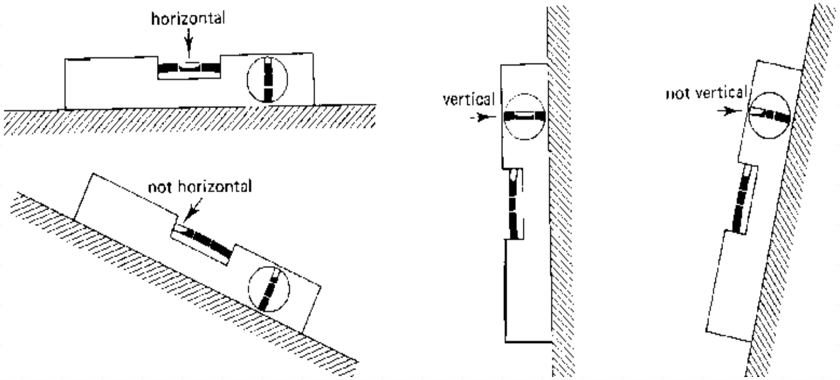


Surveying Instruments

4. Hand Level

A hand level is used to check if objects are horizontal or vertical. Within a hand level there are one or more curved glass tubes, called level tubes.

Each tube is sealed and partially filled with a liquid (water, oil or paraffin). The remaining space is air, visible as a bubble. On the glass tube there are two marks. Only when the hand level is horizontal (or vertical) is the air bubble exactly between these two marks.



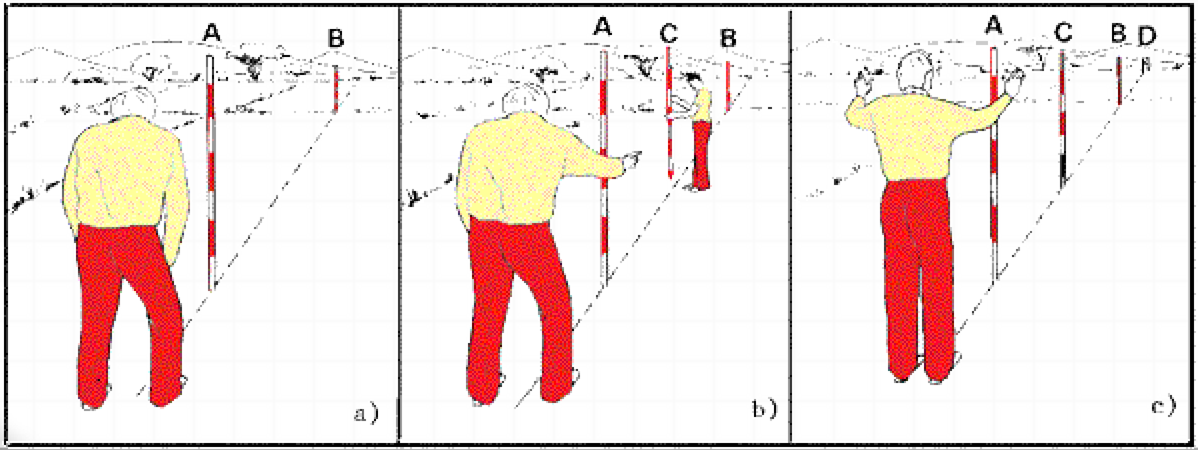
Surveying Instruments

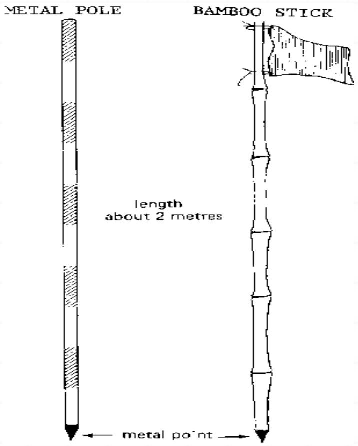
5. Ranging Poles

Ranging poles are used to mark areas and to set out straight lines on the field. They are also used to mark points which must be seen from a distance, in which case a flag may be attached to improve the visibility.

Ranging poles are straight round stalks, 3 to 4 cm thick and about 2 m long. They are made of wood or metal.

REMEMBER: Ranging poles may never be curved.
Ranging poles are usually painted with alternate red-white or black-white bands.

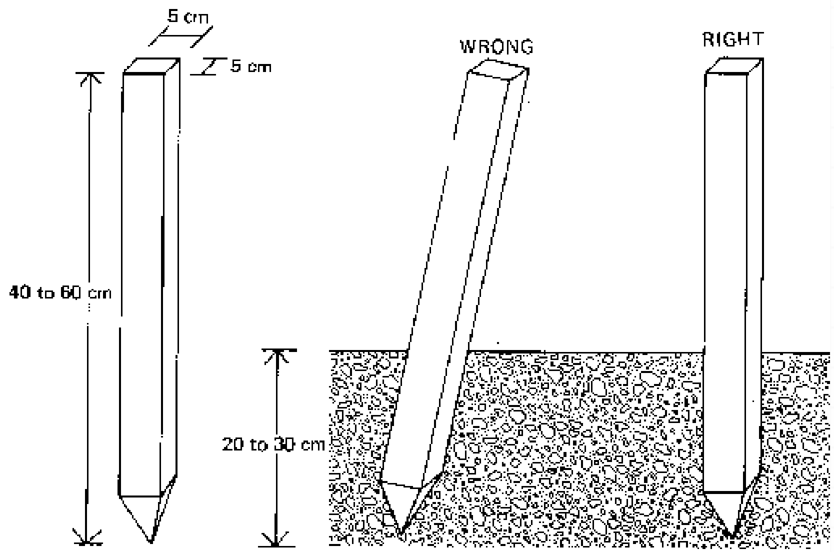


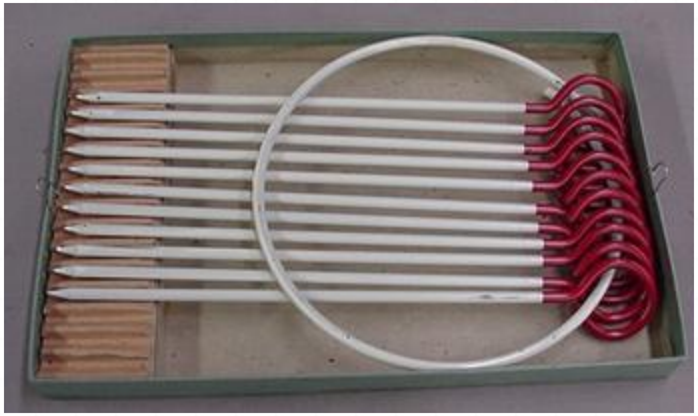


Surveying Instruments

6. Pegs and chaining pins

Pegs are used when certain points on the field require more permanent marking. Pegs are generally made of wood; sometimes pieces of tree-branches, properly sharpened, are good enough. The size of the pegs (40 to 60 cm) depends on the type of survey work they are used for and the type of soil they have to be driven in. The pegs should be driven vertically into the soil and the top should be clearly visible.



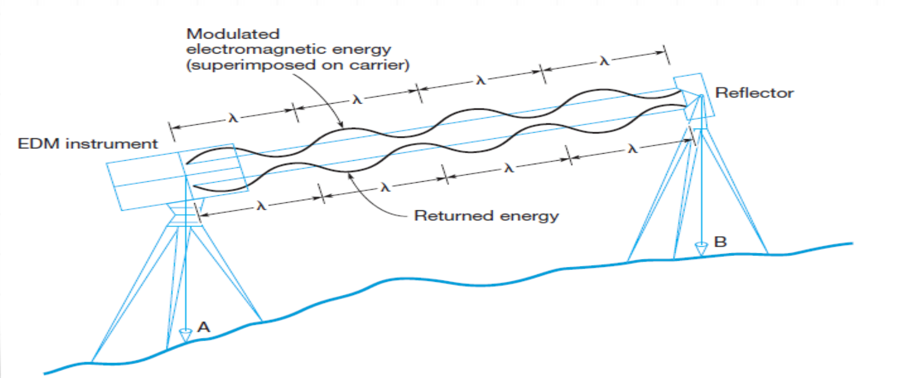


Surveying Instruments

7. Electronic distance measurement (EDM):

These devices measure lengths by indirectly determining the number of full and partial waves of transmitted electromagnetic energy required in traveling between the two ends of a line.

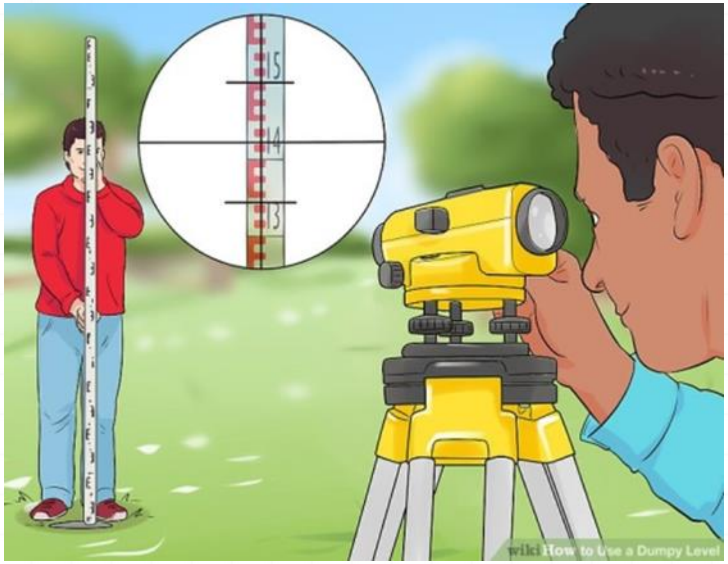
In practice, the energy is transmitted from one end of the line to the other and returned to the starting point; thus, it travels the double path distance.



Surveying Instruments

8. Levels:

Levels are used to determine elevations in a wide variety of surveying, mapping, and engineering applications



Surveying Instruments

9. Theodolites:

Theodolites (sometimes called transits) are used in measuring horizontal and vertical angles and for establishing linear and curved alignments in the field.

Surveying Instruments

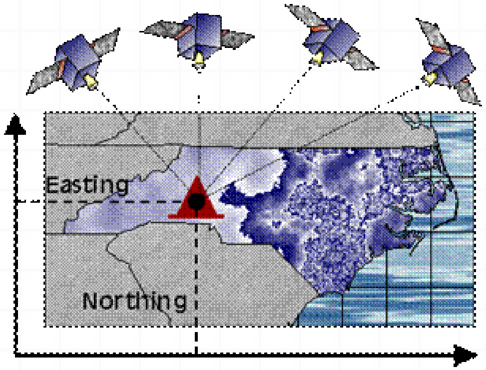
10. Total stations:

Total stations combine EDM with an electronic theodolite. In addition, it is equipped with a central processor, which enables the computation of horizontal and vertical distances. The central processor also monitors instrument status and executes software programs that enables the surveyor to perform a wide variety of surveying applications.

Surveying Instruments

11. Global Navigation Satellite System (GNSS):

Is a term used world-wide to describe the various satellite positioning systems now in use. Global positioning system (GPS) is the term used to describe the U.S. NAVSTAR positioning system, which was the original fully-operational global navigation satellite systems (GNSS). GLONASS→ the Russian GNSS, Galileo→ the European GNSS, Beidou→ China's GNSS



A satellite positioning receiver captures signals transmitted by four or more positioning satellites in order to determine position coordinates (e.g. northing, easting, and elevation) of a survey station.

Position Referencing
1- Geographic Reference

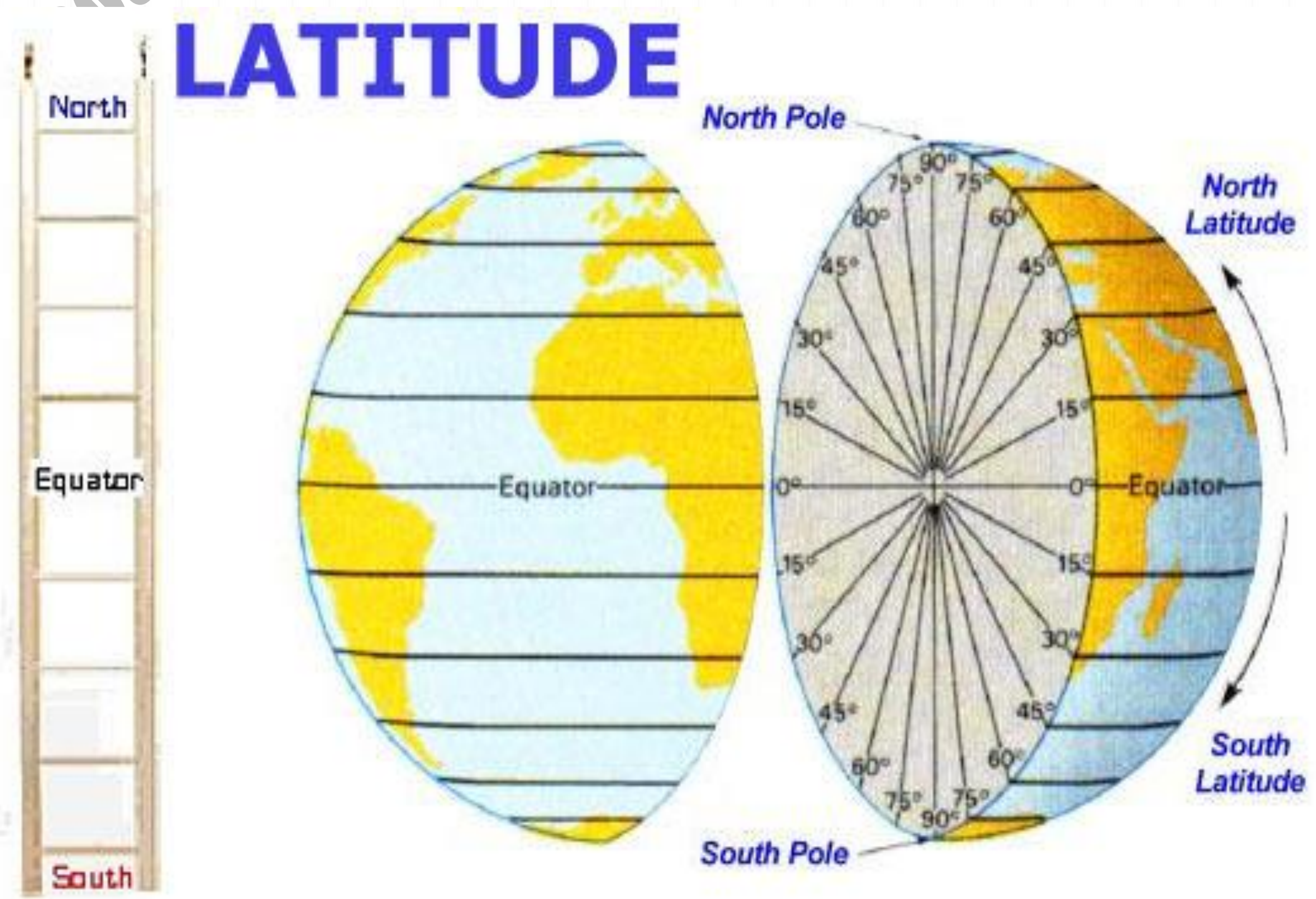
- Surveying includes measuring the location of physical land features relative to one another and relative to a defined reference on the surface of the earth.

- The earth's reference system is composed of the surface divisions denoted by geographic **latitudes** and **longitudes**

- **Latitude lines** run east/west and are parallel to the equator.

    - The latitude lines are formed by projecting the latitude angle out from the center of the earth to its surface.

    - The latitude angle itself is measured (90 degrees maximum) at the earth's center, north or south from the equatorial plane.



http://modernsurvivalblog.com/survival-skills/basic-map-reading-latitude-longitude/

Position Referencing

1- Geographic Reference

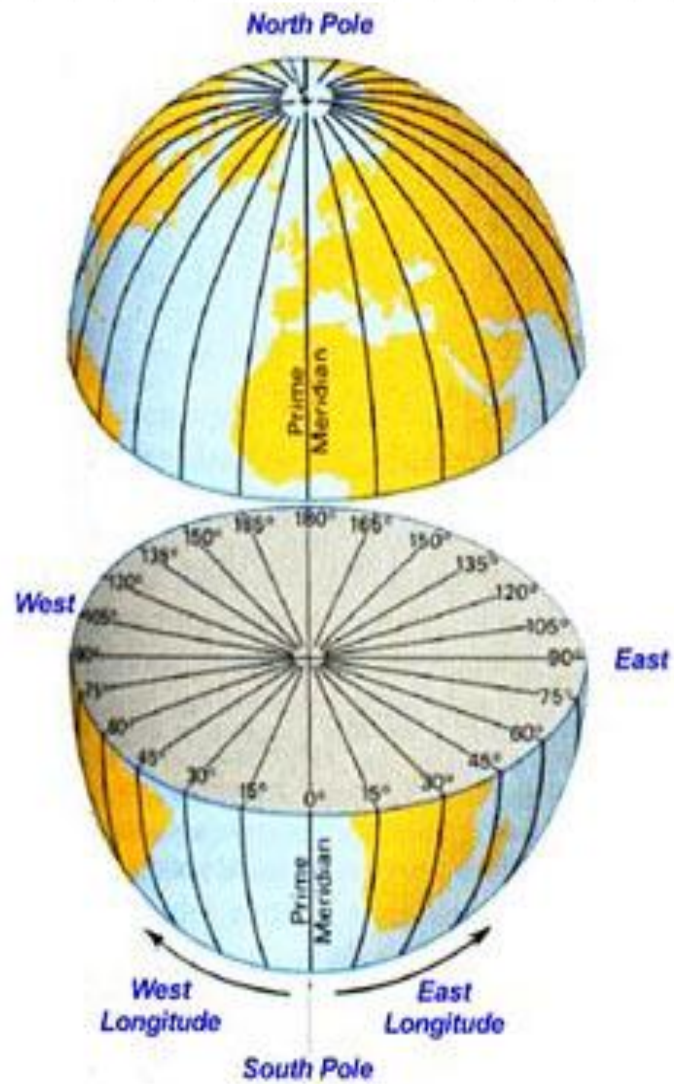
Longitude lines run north/south, converging at the poles.

- The line of longitudes (meridians) are formed by projecting the Longitude angle at the equator out to the surface of the earth .

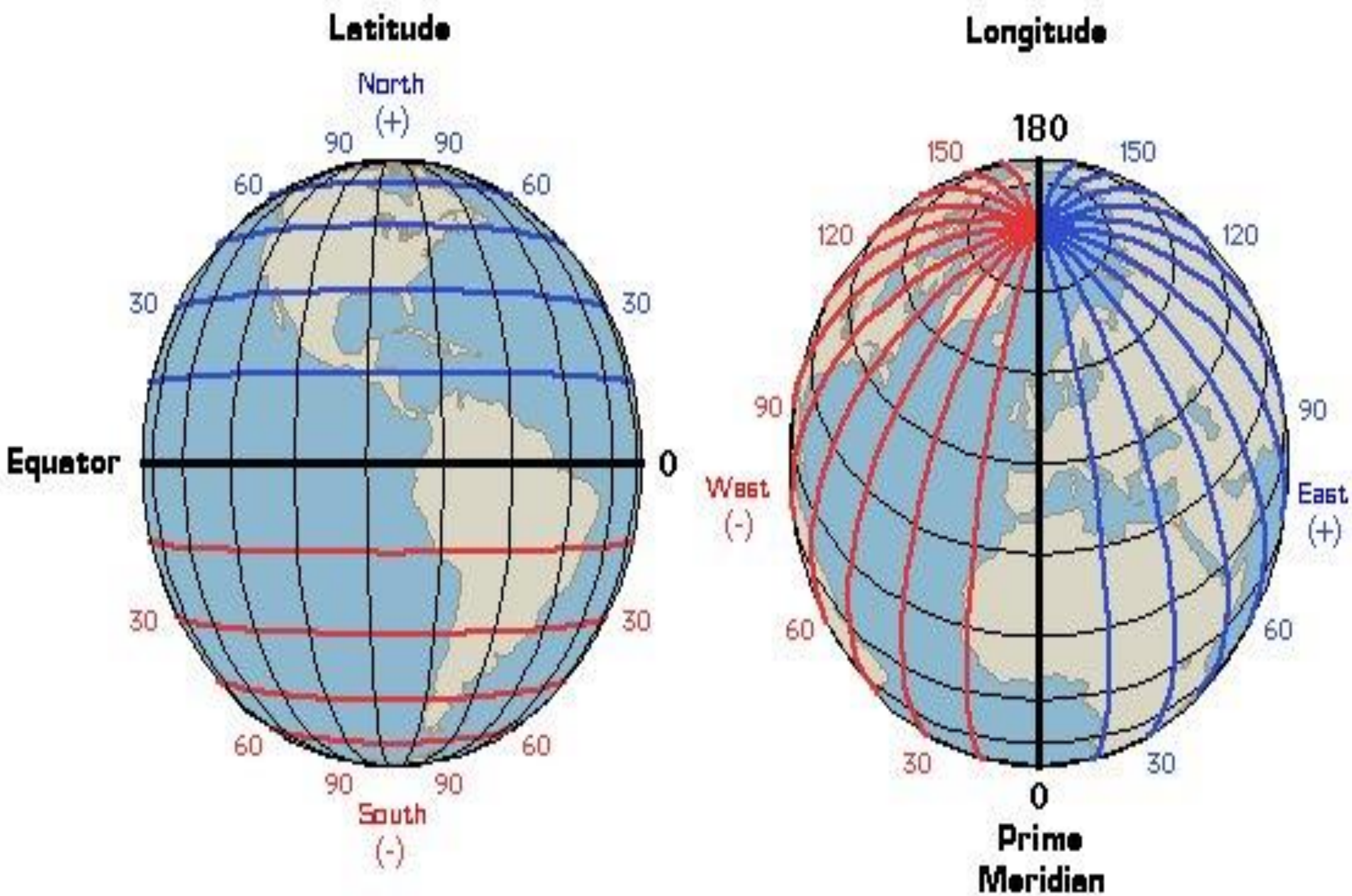
- The longitude angle itself is measured at the earth’s center, East or west (180 degrees maximum) from 0° longitude, which has been arbitrarily placed through Greenwich, England.

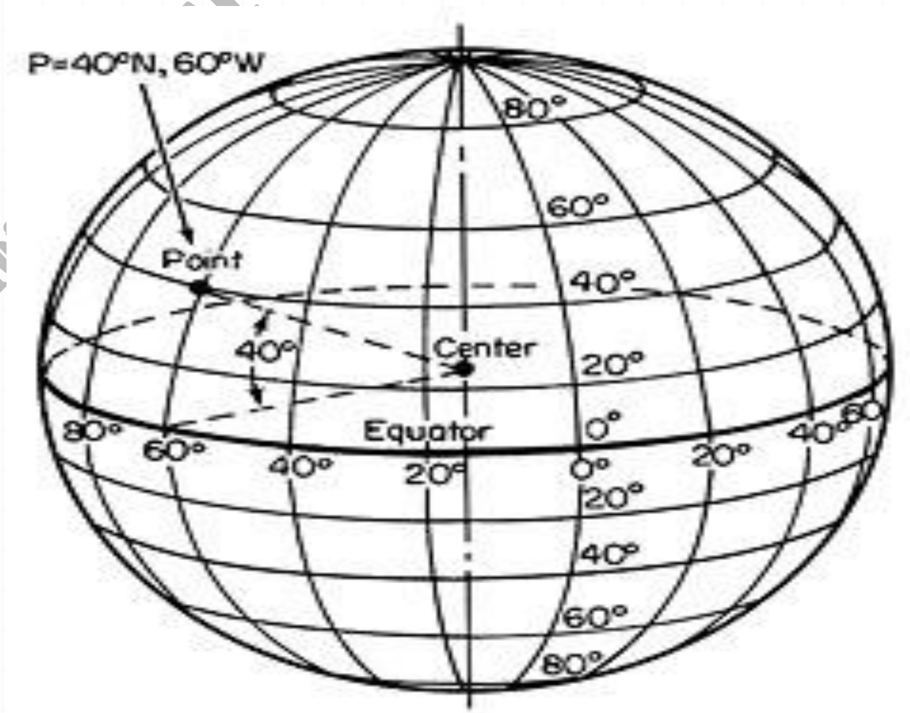
Markings of the prime meridian at the Royal Observatory, Greenwich.

LONGITUDE



http://modernsurvivalblog.com/survival-skills/basic-map-reading-latitude-longitude/


Latitude
North
(+)
90
90
60
60
30
30
Equator
0
30
30
60
60
90
90
South
(-)
Longitude
180
150
150
120
120
90
90
West
(-)
East
(+)
60
60
30
30
0
Prime
Meridian



What are the Latitude and Longitude of Amman city?

http://kaffee.50webs.com/Science/activities/Astro/Activity-Latitude.Longitude.htm

Position Referencing
2- Grid Reference

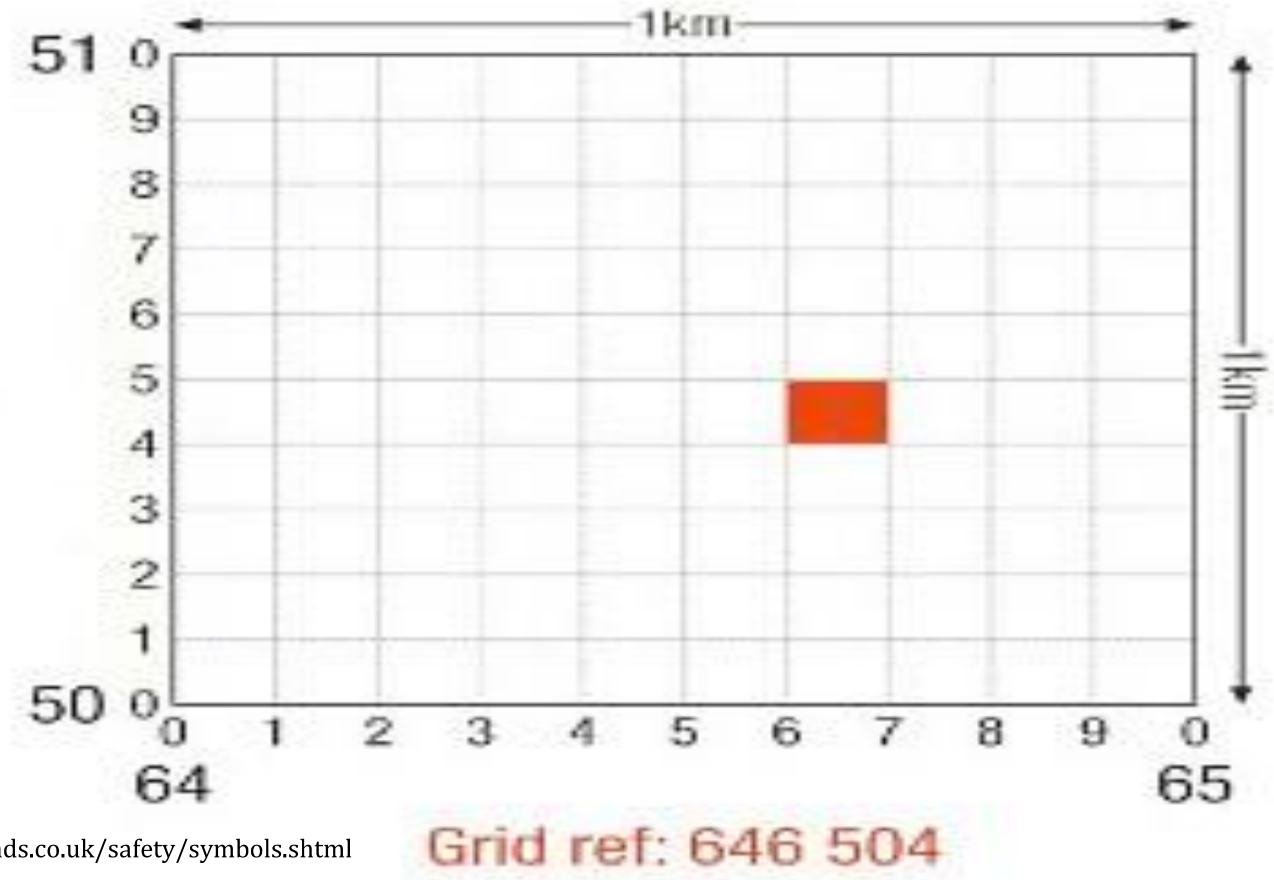
- Grid references define locations on maps using **Cartesian coordinates.**
- Grid lines on maps define the coordinate system, and are numbered to provide a unique reference to features.
- Grid systems vary, but the most common is a **square grid with grid lines intersecting** each other at **right angles and numbered sequentially from the origin at the bottom left of the map.**
- The grid numbers on the **east-west (horizontal) axis are called Eastings, and the grid numbers on the north-south (vertical) axis are called Northings.**

Position Referencing
2- Grid Reference

- Numerical grid references consist of an **even number** of digits. **Eastings are written before Northings**. Thus in a 6 digit grid reference 123456, the Easting component is 123 and the Northing component is 456.
- The grid is limited in area e.g. 1 squared. Km, no serious errors resulting from ignoring curvature
- Ease of calculations (plane geometry and trigonometry)
- Translation to geographic coordinates could be accomplished.

Position Referencing

2- Grid Reference



http://www.walkhighlands.co.uk/safety/symbols.shtml

Position Referencing
3- Vertical Reference

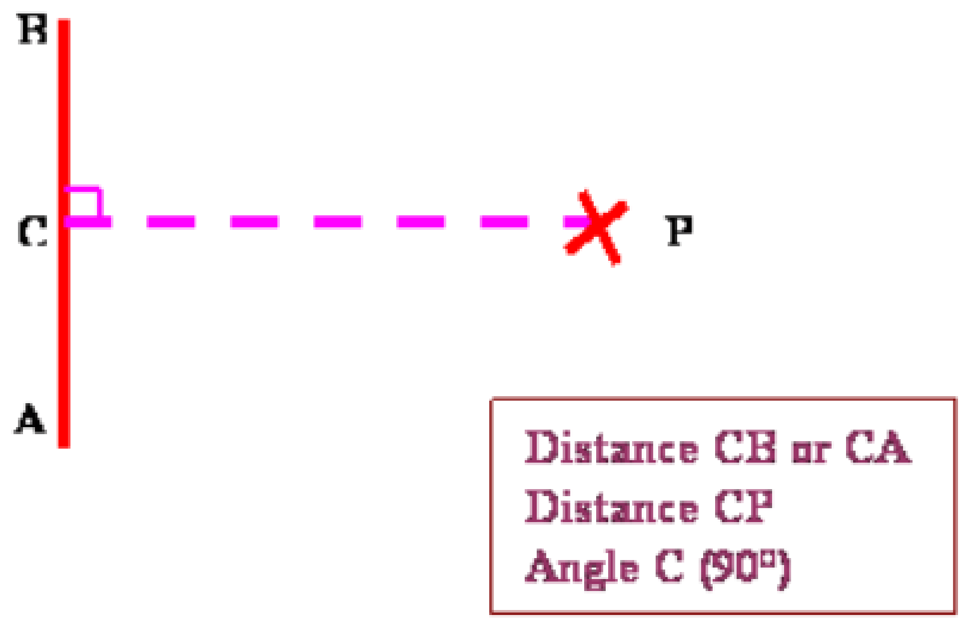
In addition to the X and Y dimensions of any feature, a vertical dimension can be referenced to any datum, usually Mean Sea Level (MSL).

Locating Point Reference to a Line

Point "P" may be located relative to line AB by:

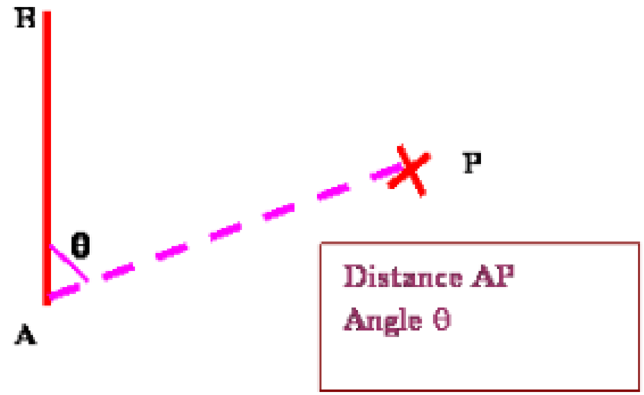
- Line AB is previously known
- Required to locate point P relative to line AB

1: Right-Angle offset tie (rectangular tie):

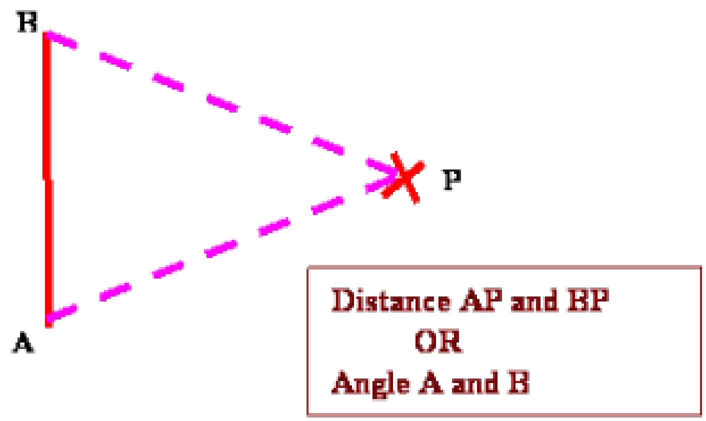


Locating Point Reference to a Line

2: Angle-Distance tie (polar tie):



3: Intersection technique:



Accuracy & Precision

- *Accuracy:* is the relationship between the value of a measurement and the "true" value of the dimension being measured

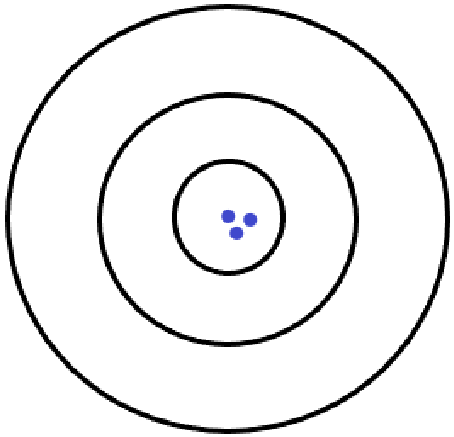
- *Precision:* describes the refinement of the measuring process and the ability to repeat the same measurement with consistently small variations in the measurements

- Ex: a wall known to be 157.22 ft long is measured using two tapes; types

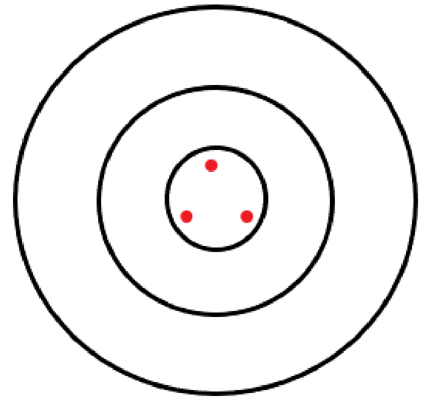
	"True" Distance (ft)	Measured distance (ft)	Error (ft)
Fiberglass tape	157.22	157.3	0.08
Steel tape	157.22	157.23	0.01

Accuracy & Precision

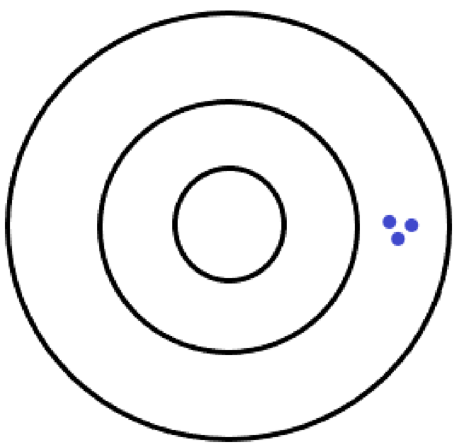
High Accuracy/ High precision



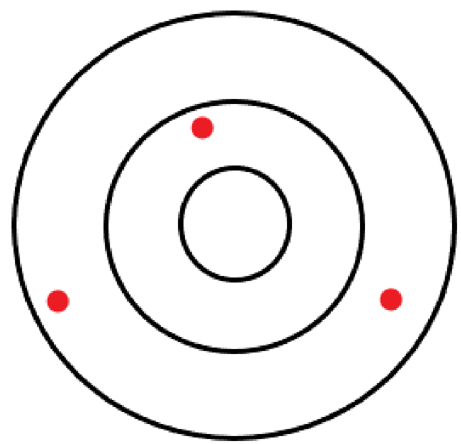
High Accuracy/ Low precision



Low Accuracy/ High Precision



Low Accuracy / Low precision



Accuracy & Precision

Accuracy Ratio: is the ratio of error of closure to the distance measured

The error of closure: is the difference between the measured value and the theoretical correct value.

- Ex: a distance was measured and found to be 250.56 ft. the distance was previously known to be 250.50 ft. the error is 0.06 ft in a distance of 250.50 ft.

- Accuracy ratio (AR)= 0.06/ 250.50
  = 1/ 4175 ≈ 1/4200
- AR is expressed as a fraction whose numerator is unity and whose denominator is rounded to closest 100 units

- ***Practice: If the measured internal angles in a triangle are: 71º 12' 13", 55º 34' 27", and 53º 56' 37", find the error of closure and the accuracy ratio in the measurements.***

Errors

1. **Systematic errors:** errors whose magnitude and algebraic sign can be determined.
   - surveyor can eliminate them to improve the accuracy
   - Example: effect of temperature on steel tape

2. **Random errors (accidental error):** Occurs in every surveying measurement and is beyond the control of the surveyor. It is due to the nature of human being.

- If surveyor is skilled and careful, random error will not be significant.

Mistakes

Mistakes are blunders made by a survey personnel.

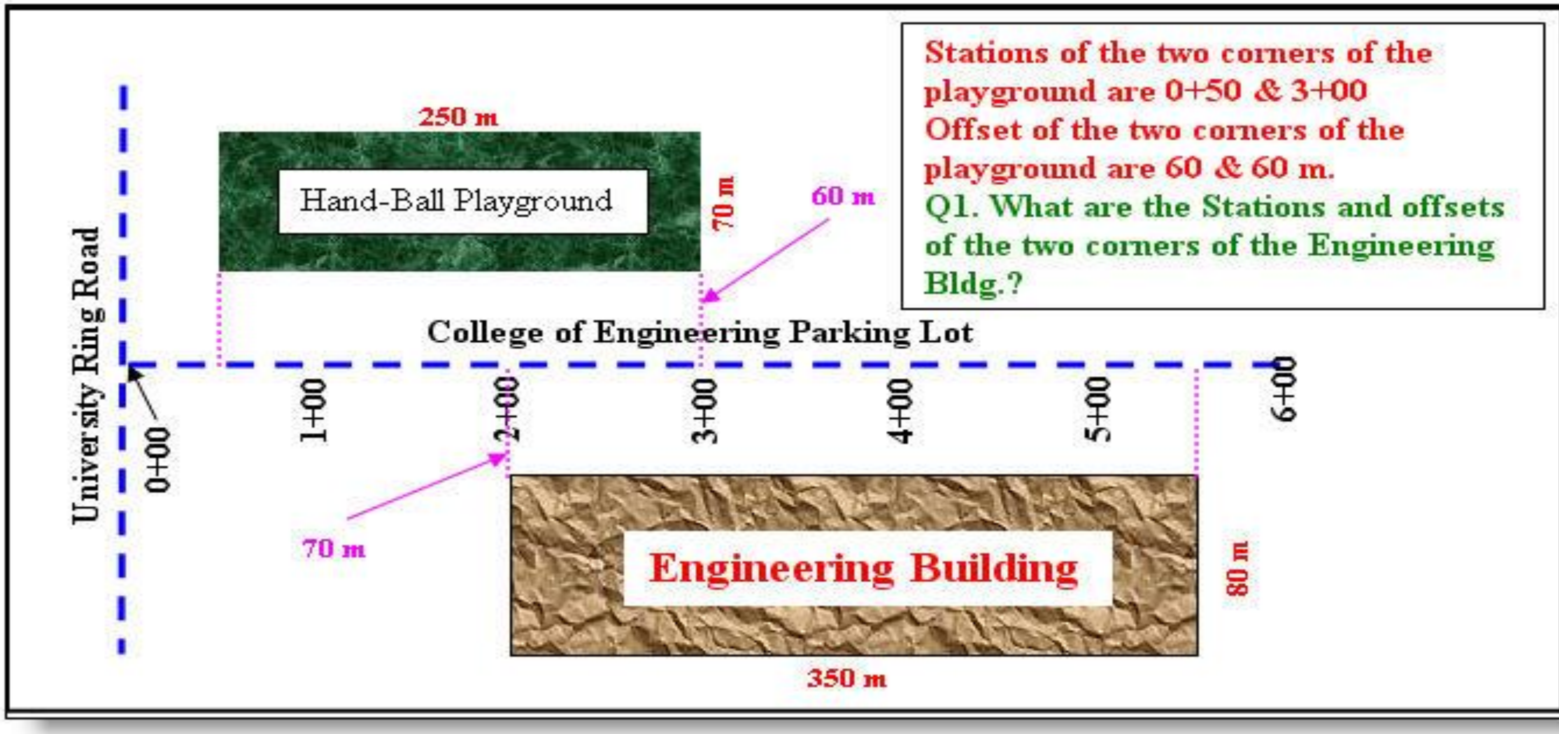
Example:

- transposing figures (recording a tape value of 68 as 86)
- Measuring from and to the wrong point

- They should be discovered and eliminated, by verification (repeating the measurement, or geometric or trigonometric analysis).

Stationing

- Distances along baseline are called: Stations or Chainage.
- Measurements at right angles to baseline are called: Offsets.



Stationing

Many highway agencies use 1000 unit station 1+000

⇒ Full station	100 m	or 100 ft
⇒ Half station	--	50’
⇒ Partial station	20 m	--

- Example: If the station of a certain position is 2 + 36.72 and stations are taken each 100 m. Then the distance of that position from the base point is
- = 200 + 36.72
- = 236.72

Field Management

- **<u>Survey Crew</u>**
- **Chief:** responsible about the whole work.
- **Instrument operator:** operation and care of the instrument.
- **Survey assistance:** perform taping and carry the rods and prisms.

- **<u>Field Books</u>**
- **Bound books.**
- **Loose leaf books.**
-
- Include name and address in the first page.
- Right pages are kept for the sketches.
- Pages should be numbered.
- Should show project name, number, weather, date, used instruments at the first page of the project (lab).

- **<u>Field Notes</u>** should be" CAN " Complete, Accurate, & Neat

Prof. TALEB AL-ROUSAN

Surveying

2104011365

Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University

Prof. TALEB AL-ROUSAN

Chapter 2
Tape Measurements

Methods of Horizontal Distance Measurement

- Pacing
- Car Odometer & Measuring wheel
- Gunter's Chain
- Tape
- Stadia (Tacheometry)
- Subtense bar
- Electronic Distance Measuring (EDM)

Pacing

- Imprecise
- Medium (neither long or short) .
- Pace length depends on legs opening.
- Very useful
- Accuracy on level ground 1/50 to 1/100
- Inaccurate in hilly areas

Note: One pace = perimeter in m/ perimeter in pace

Car Odometer & Measuring Wheel

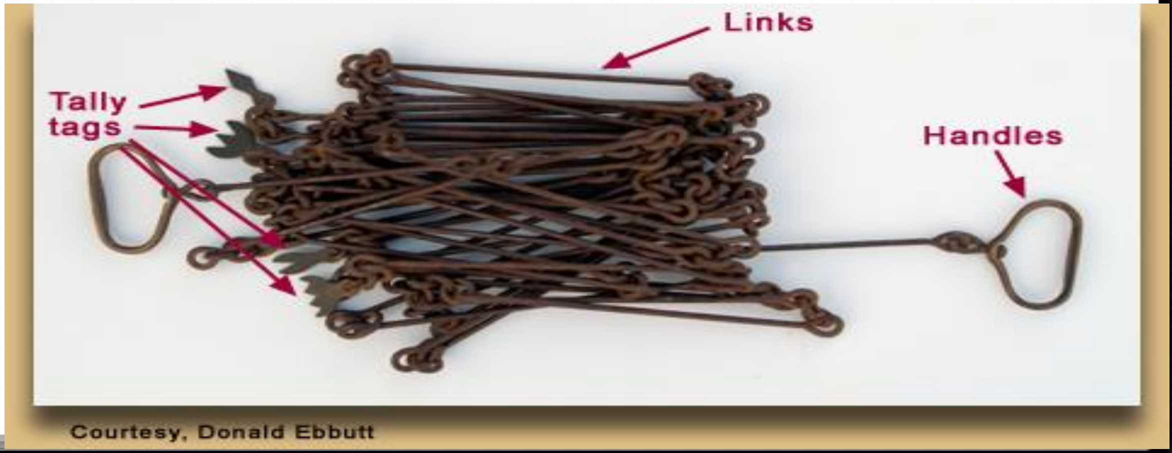
- Accurate enough to collect information to begin a survey.
- Fence lines abutting a road
- Distance to a traffic accident scene
- One rev $=2\pi r$

Then, n rev = d

then $d = 2\pi r n$

Gunter's chain

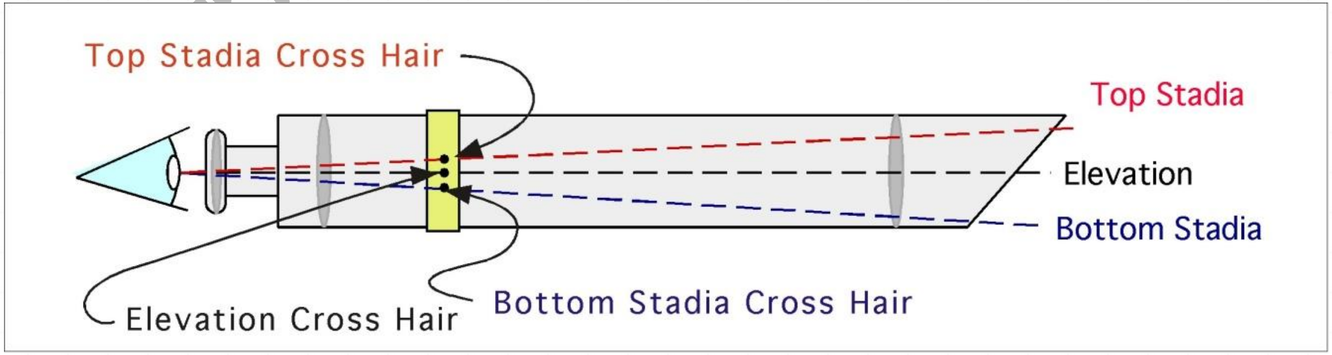
- The Gunter's chain used to survey North America was 66 ft long and composed of 100 links (chain=100 link)
- Very old
- Not used anymore
- 80 chains = 1 mile
- 10 square chain = 1 acre



Tape

- Traditional method of measuring distance.
- Available in steel, cloth, and fiberglass (good when working near electric installation)
- Steel tapes are available in 20, 30, 50 m (100, 200, and 300 ft).
- Cross section: light duty (6 mm * 0.3 mm) and heavy duty (8 mm * 0.454 mm) which can be dragged.

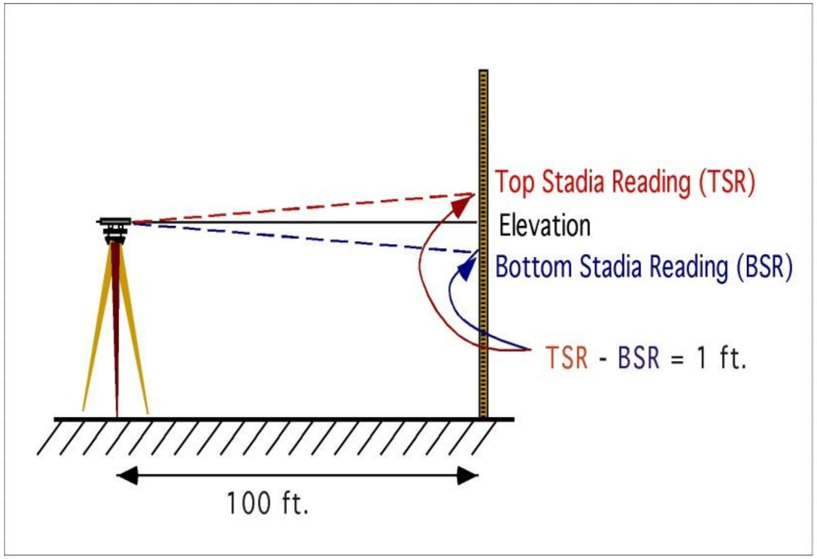
Stadia (Tacheometry)



Distance by stadia requires an instrument with stadia cross hairs.

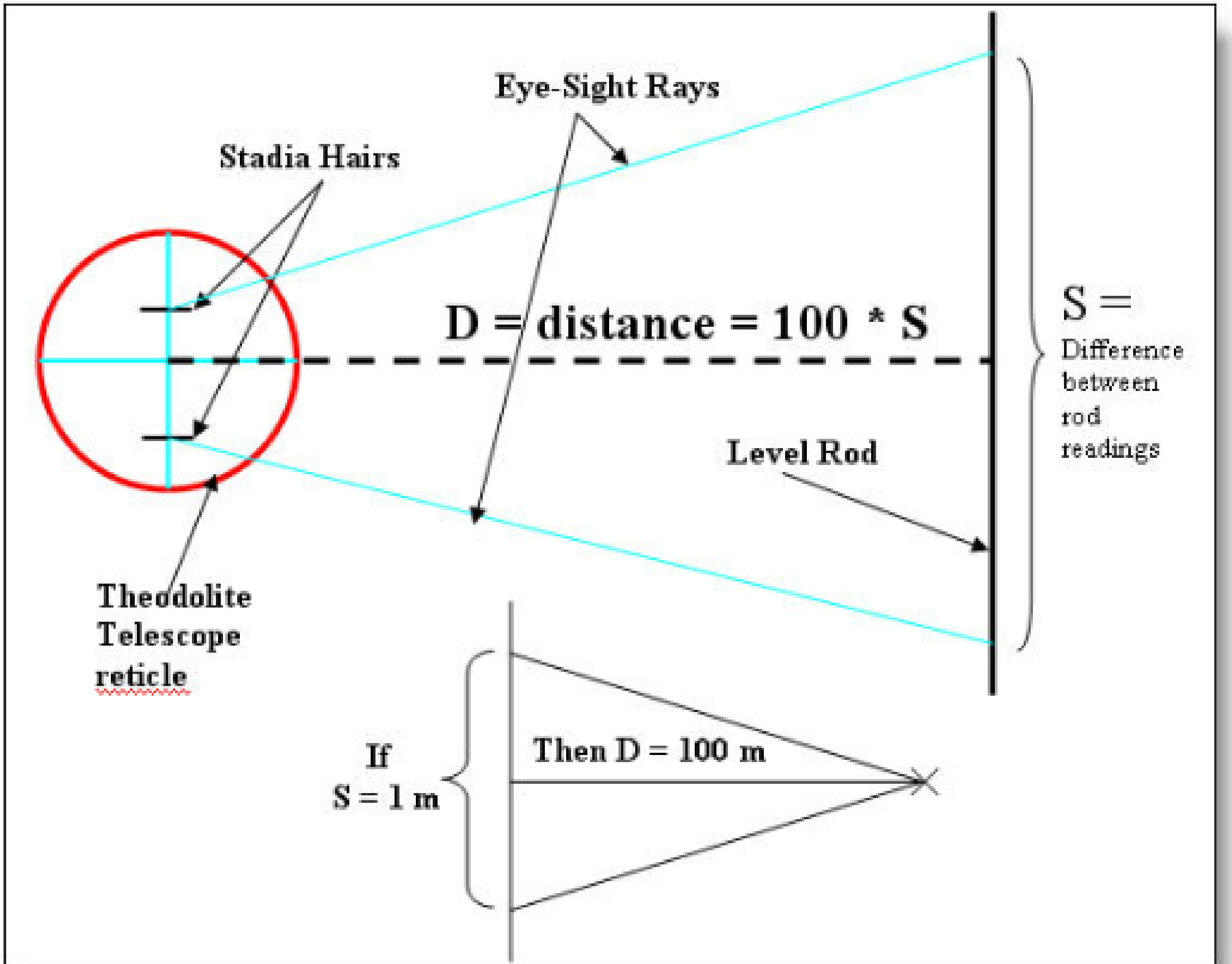
The distance between the stadia crosshairs is designed so that the divergence of the sights across the two stadia crosshairs is 1.0 feet when the instrument is 100 feet from the rod.

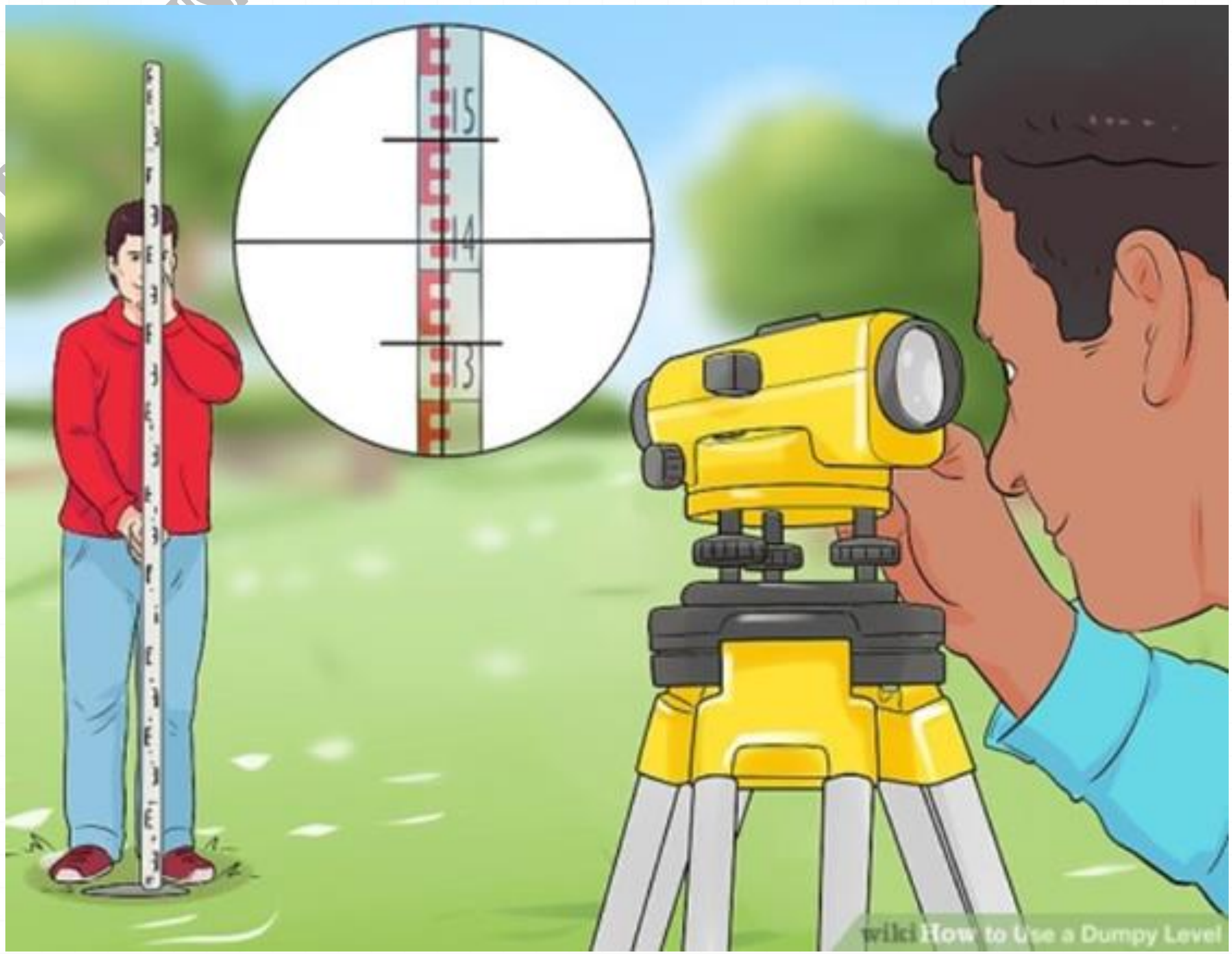
(Assuming an instrument stadia factor of 100.)



Stadia

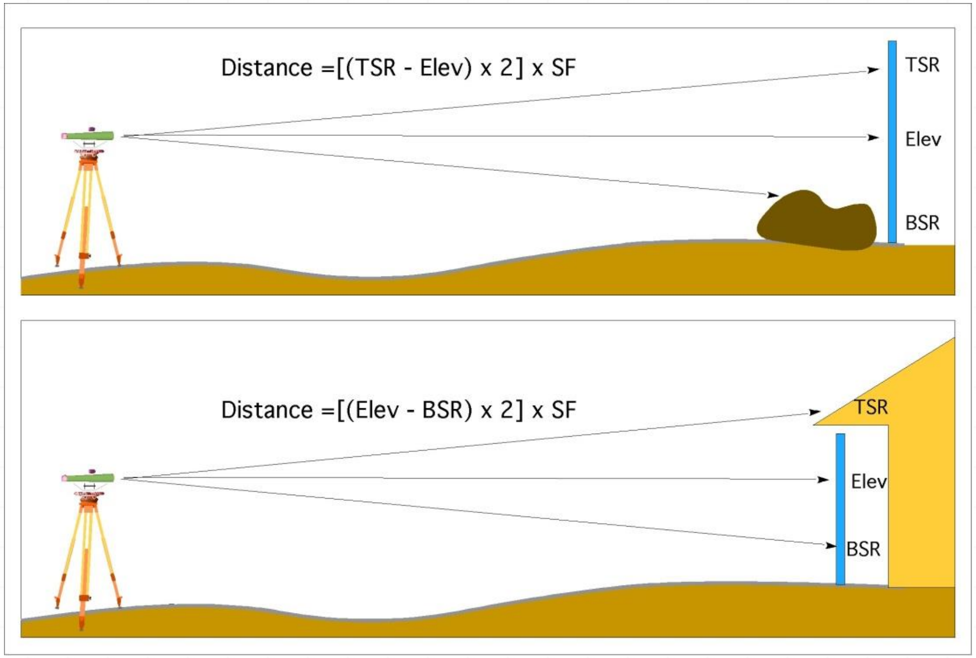
- Tachometry technique (uses trigonometry calculation)
  - Uses the horizontal marks on the theodolite cross-hair




15
14
13
wikiHow to Use a Dumpy Level

When the top or bottom stadia hair rod reading is obscured, a process called 1/2 stadia can be used. When 1/2 stadia is used the elevation crosshair, and which ever stadia crosshair that can be read, is used.

Because this stadia interval is 1/2 of the standard interval, it is multiplied by two.



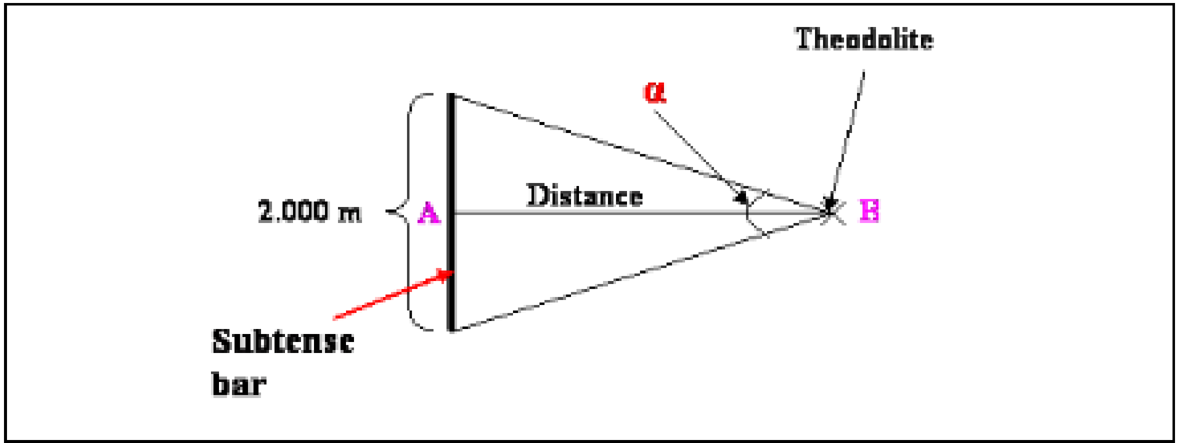
$$\text{Horizontal Distance} = \left[\left(\text{TSR} - \text{Elev}\right) \times 2\right] \times 100$$

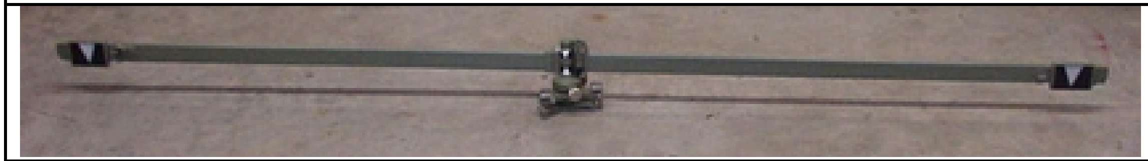
$$= \left[\left(7.34 - 6.21\right) \times 2\right] \times 100$$

$$= 226 \text{ ft}$$

Subtense Bar

- Dist T_1 - T_2 = 2 m regardless of Temp.
- Need a theodolite.



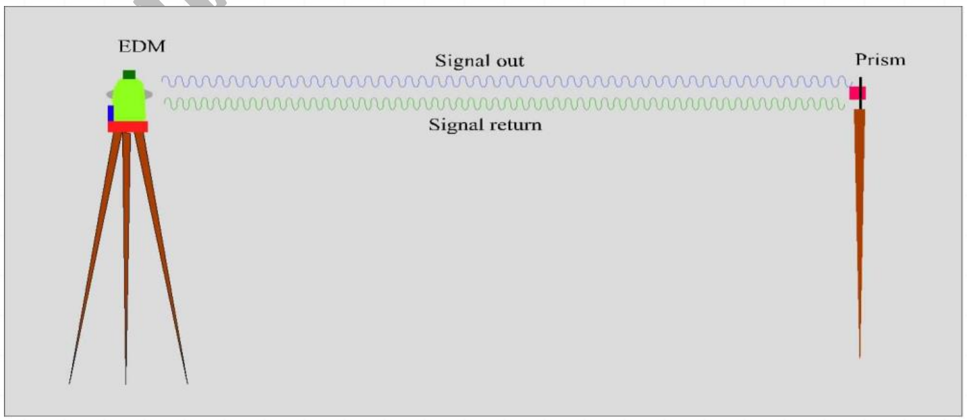


$$\tan \alpha/2 = 1 / AB$$

$$\text{Distance AB} = \frac{1}{\tan \alpha/2} = Cot\,.\,\frac{\alpha}{2}$$

- α is independent of any vertical angle.
- Very good for hilly or mountainous country
- Not very accurate for long distances.

Electronic Distance Measuring (EDM)



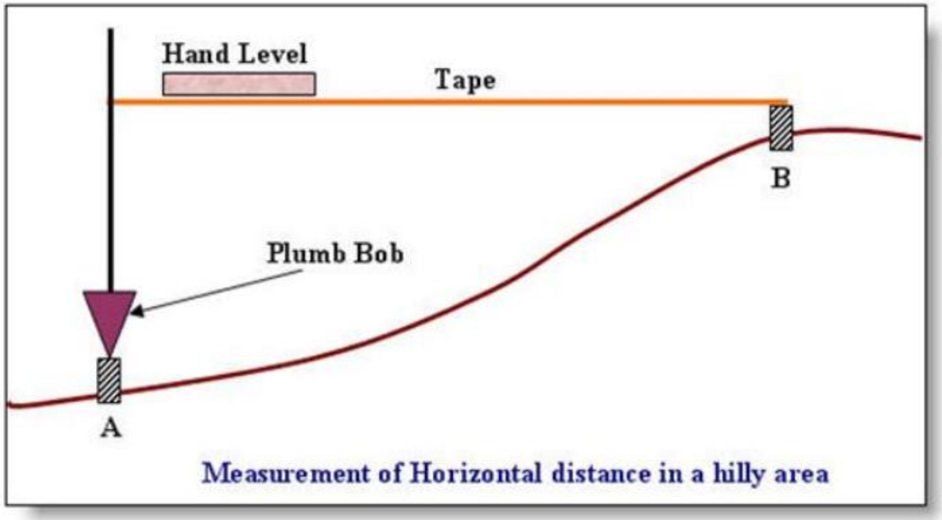
- The term EDM is used to describe a category of instruments that measure distance using an electronic signal.
- The instrument broadcasts a focused signal that is returned by a prism or reflection from the object.
- Very accurate
- Using light wave or microwave
- By measuring phase difference between transmitted and received signals

Taping Accessories

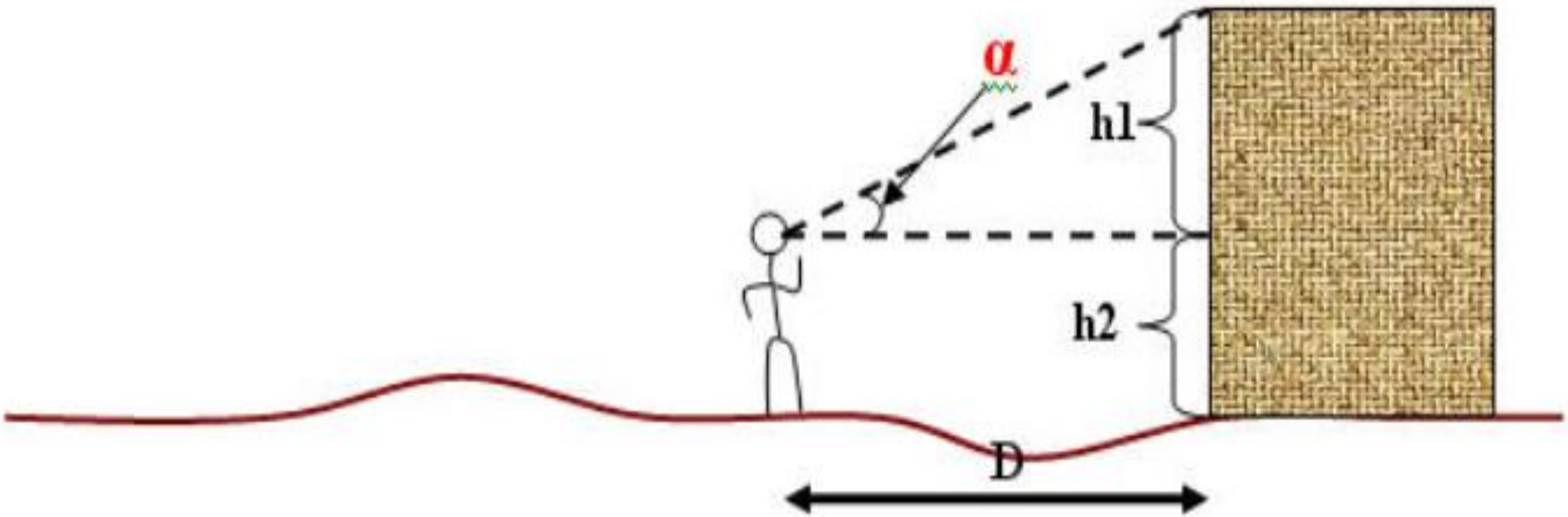
Plumb Bob:
Used to transfer from top to ground when top is held off ground to maintain the tape horizontal.

Hand level:
1-Normal hand level
- for horizontal level determination
- To keep tape horizontal

2- Abney hand level (Clinometer)
- for horizontal level determination
- for vertical angle and slope
- for height determination (not very accurate)



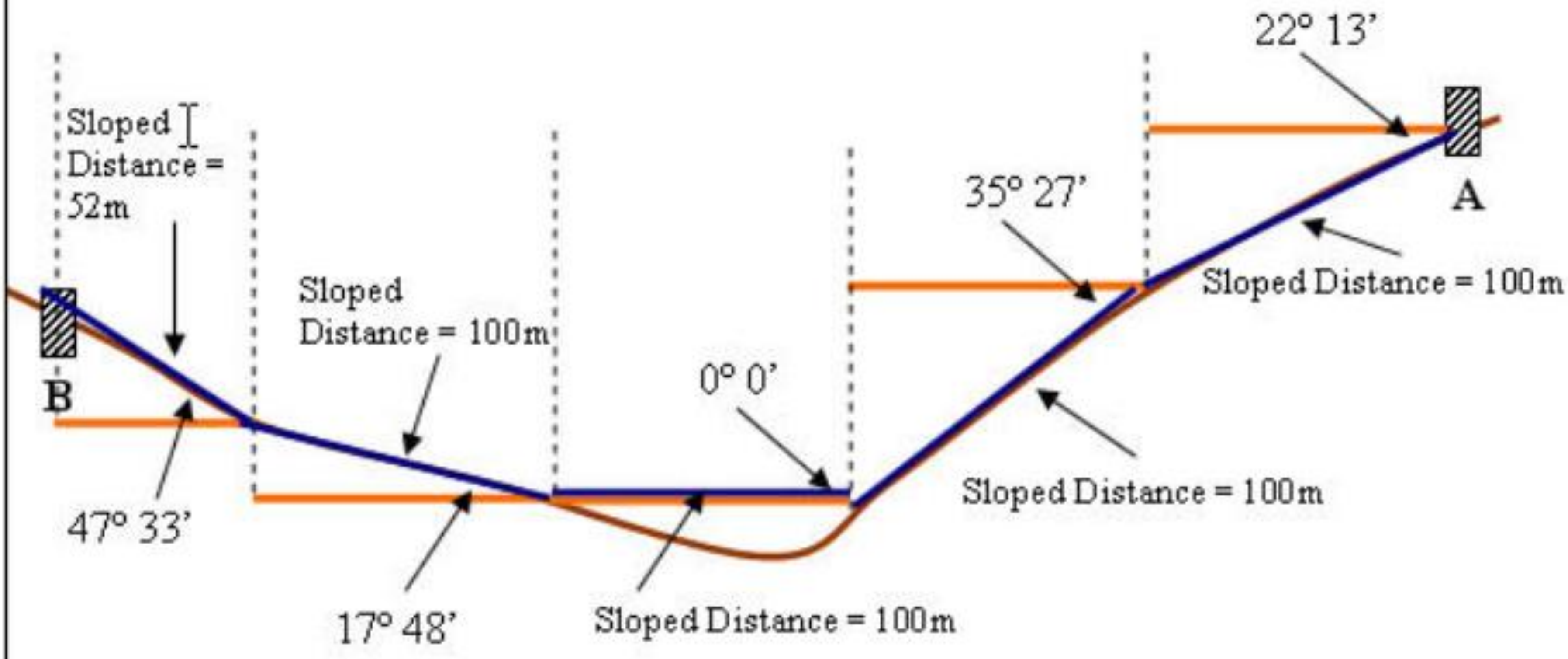
Taping Accessories/ Clinometer



If the height of the eyesight of the surveyor is 1.62 m,the measured angle by the Clinometer is 35°, and the measured horizontal distance between the surveyor and building (D) = 10.53, find the total height of the building.

Measurement of height building using the Abney Hand Level

Taping Accessories / Clinometer



Find the total horizontal distance between points A & B.

Getting the horizontal distance between two points using the tape to measure the sloped distances and the Clinometer to measure drop angles.

Taping Accessories

Plumb bob target

Tension handle

Taping Methods

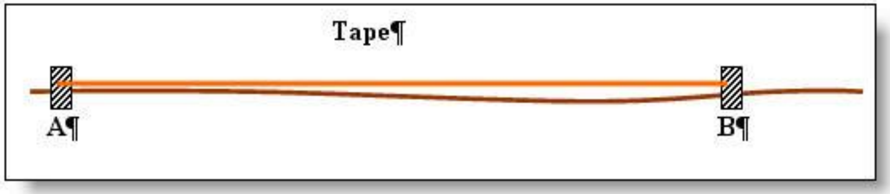
Normally the tape is held horizontally. Therefore, the way the tape is held depends on land topography.

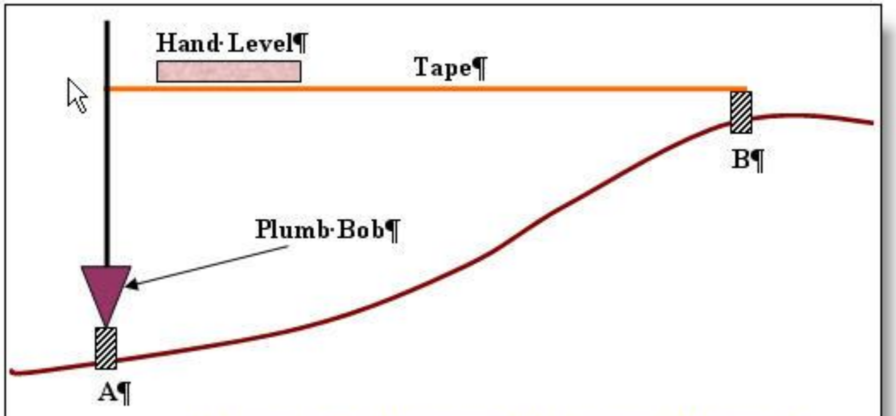
There are three methods of holding the tape.

1- On smooth leveled land, the tape can be laid on the ground.

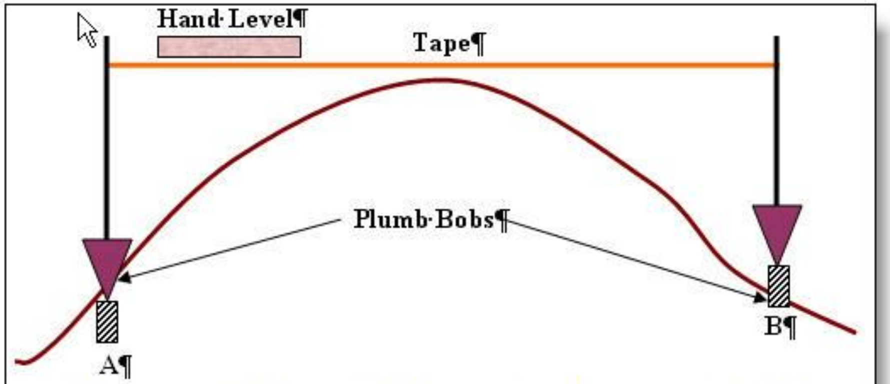
2- On sloping lands, need to use a plumb bob. For extra accuracy, a hand level and tension handles are required.

3- If there are obstacles, two plumb bobs are required





Measurement·of·Horizontal·distance·in·a·hilly·area¶



Measurement·of·horizontal·distance·when·there·is·an·obstacle.¶

Taping Procedures

1- Head surveyor holds the zero end of the tape and starts walking towards the target.

2- Back surveyor shouts “tape” when the full length of the tape is unwound.

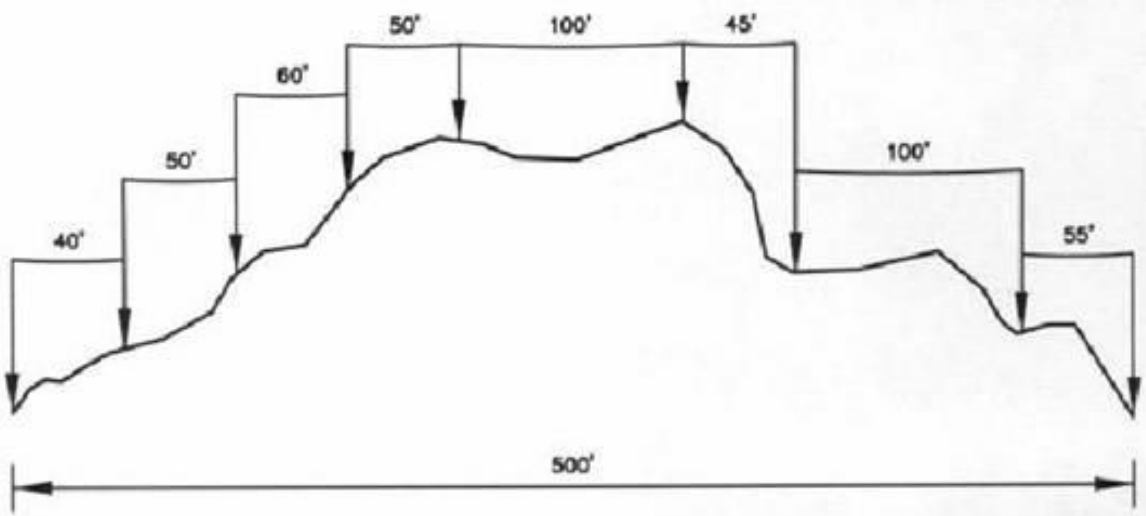
3- Rear surveyor aligns the front surveyor depending on a fixed range pole at the end of the distance.

4- After applying the required tension, the front surveyor places a chaining pin in the ground.

5- Both surveyors repeat the procedure, and the rear surveyor collects the pins after finishing from that point.

Taping in Practice

- Usually horizontal distance is measured directly whenever possible.
- If distance is large the Breaking-Tape technique is used.
- For preliminary route survey using a 100 m tape, the slope distance is measured using the tape, and the slope angle is measured using the Abney level.



Errors

1. **Systematic errors:** errors whose magnitude and algebraic sign can be determined.
   - surveyor can eliminate them to improve the accuracy
   - Example: effect of temperature on steel tape

2. **Random errors (accidental error):** Occurs in every surveying measurement and is beyond the control of the surveyor. It is due to the nature of human being.

   If surveyor is skilled and careful, random error will not be significant.

Taping Errors:

Systematic	Random
Slope	Slope
Erroneous Length	Temperature
Temperature	Tension and sag
Tension and Sag	Alignment
	Marking and plumbing

Q1. Why are some of the errors included in both systematic and random errors?

Tape standard Conditions

Foot System (100 ft) steel tape	Metric system (30 m) steel tape
temperature= 68 F	Temperature= 20 C
Tape fully supported	Tape fully supported
Tape under a tension of 10 lbs	Tape under a tension of 50N

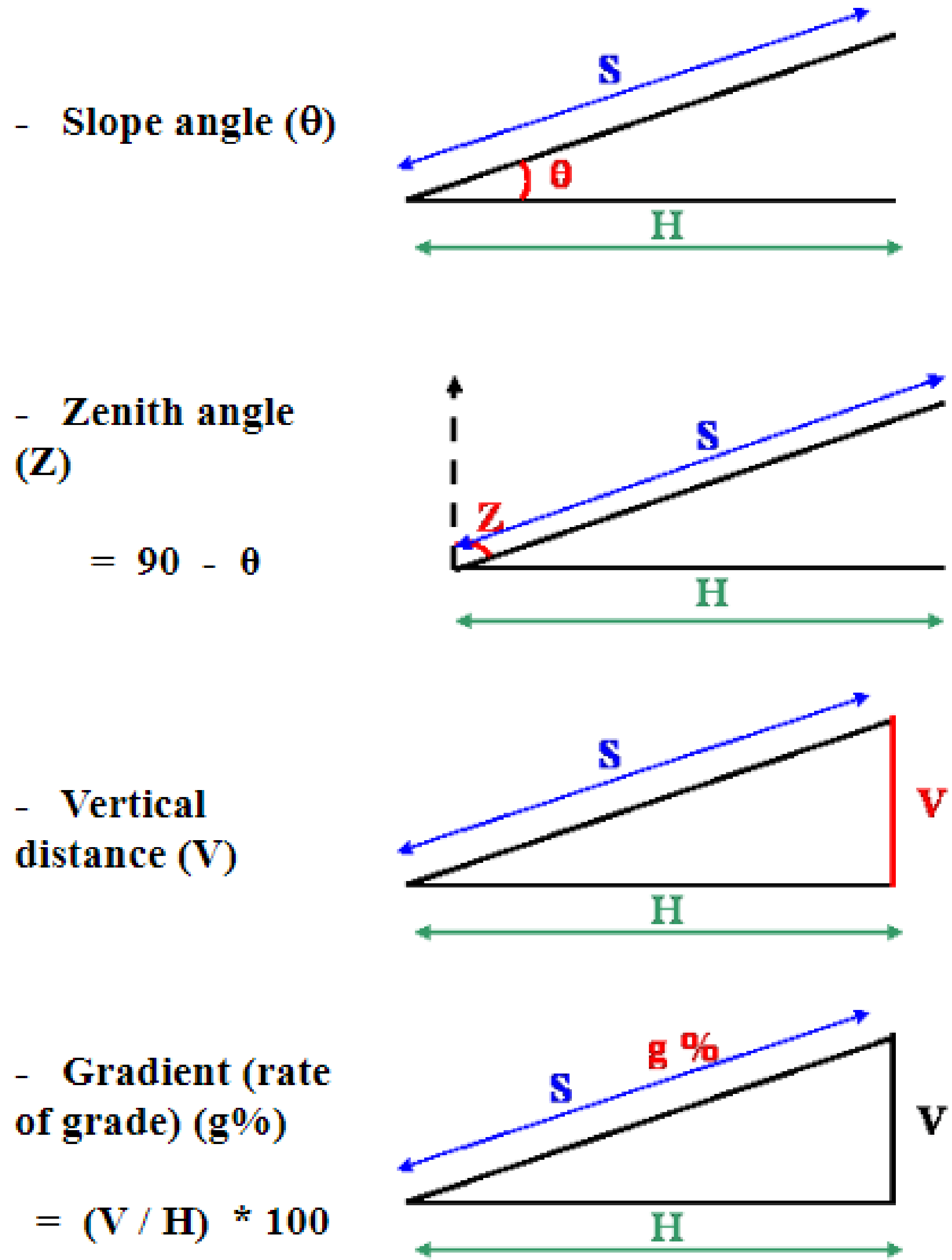
Slope Corrections

- If measurements were taken on a slope, they must be converted to their horizontal equivalent distance.

- Using either the slope angle or the vertical distance.

- Slope might be expressed as a gradient or rate of grade

· To convert you should have one of the following:



Example 1. Given Station of point A = 2 + 25

Elevation of point A = 228.32 m

Station of point B = 7 + 32

AB gradient = - 2.5 %

Required Elevation of point B.

Solution :

H = 732 – 225 = 507 m

g = (V / H) * 100

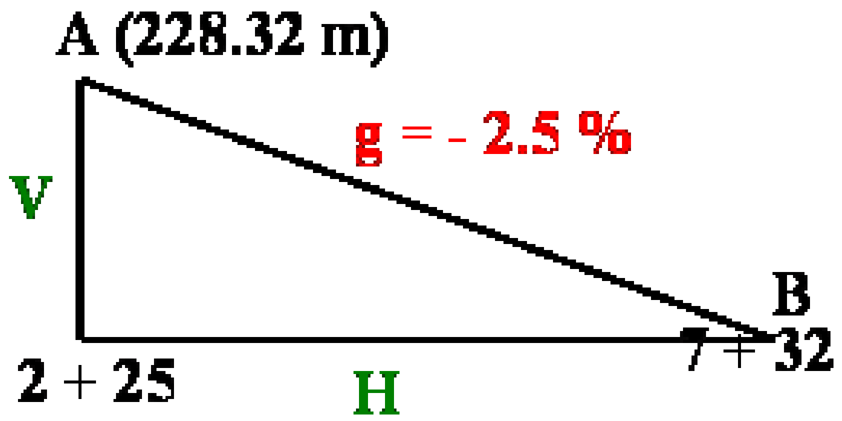
-2.5 = (V / 507) * 100

V = -0.025 * 507

= -12.68 m

B elevation = 228.32 – 12.68

= 215.64 m



Example 2. Given Station of point A = 5 + 275

Elevation of point A = 375.85 m

Station of point B = 23 + 045

Elevation of point B = 123.67 m

Required Gradient of line AB.

Solution :

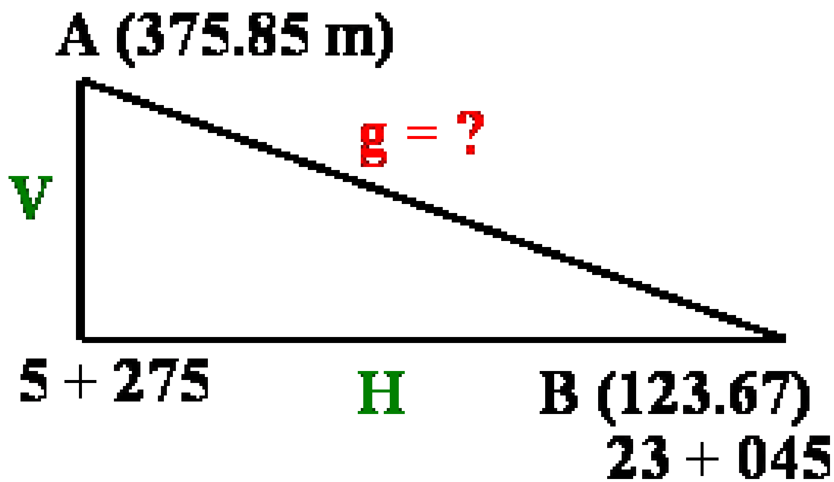
H = 23045 – 5275 = 17770 m

V = 375.85 – 123.67 = 252.18 m

g = (V / H) * 100

g = (252.18 / 17770) * 100

Gradient of line AB = 1.42%



Find the error in my solution.

Tape Length Corrections

- Through extensive use of tapes, they become kinked, stretched and repaired. The length can become something other than the specified length. In this case, correction has to be made.

Correction per tape length = Actual length – Seen Length

Find the number of times the tape was used = (Distance/seen length of tape)

Total correction = (Correction per tape length * number of times the tape was used)

Corrected measured distance = (measured distance + total correction)

In the case of laying out distance on the ground

Corrected distance = (required distance - total correction)

Tape Length Corrections

Example 1. Given Measured distance = 171.278 m

Used tape 30 m (seen)

Actual length = 29.996 m (actual)

Required Corrected length.

Solution : Correction per tape length = Actual length – Seen Length

= 29.996 – 30.000 = - 0.004 m

Number of times the tape was used = 171.278 / 30 = 5.709

Total correction = - 0.004 * 5.709 = - 0.023 m

Corrected measured distance = 171.278 - 0.023 = 171.255 m

Tape Length Corrections

Example 2. Given Distance to lay out = 210.08 m

Used tape 30 m (seen)

Actual length = 30.006 m (actual)

Required Correct length to be laid out.

Solution : Correction per tape length = Actual length – Seen Length

= 30.006 – 30.000 = + 0.006 m

Number of times the tape will be used = 210.080 / 30 = 7.003

Total correction = + 0.006 * 7.003 = + 0.042 m

Corrected distance to be laid out = 210.080 - 0.042 = 210.038 m

Temperature Corrections

⇒ Standard temp for <u>steel</u> tape:

68°F or 20°C

⇒ Thermal coefficient of expansion for <u>steel</u> tape:

(0.00000645 /ℓ.°F)

or

(0.0000116 /ℓ.°C)

⇒ Correction due to Temp.

$$C_t = a * (T - T_s) * L$$

where,

C_t ⇒ temperature correction

a ⇒ thermal coefficient

T ⇒ Temp. of tape

T_S ⇒ Standard temp

L ⇒ Total Distance measured

Temperature Corrections

Example 1. Given Distance to lay out = 210.08 m
Used tape 30 m (seen)
Tape temperature will be 27°C

Required Correct length to be laid out.

Solution : Temperature Correction = $C_t = a * (T - T_s) * L$

$$= 0.0000116 * (27 - 20) * 210.08 = + 0.017 \text{ m}$$

Corrected distance to be laid out = 210.080 - 0.017 = 210.063 m

---

⇒ For normal work ⇒ Air temp will be sufficient.

⇒ For accurate work of (1/10,000)accuracy ⇒ use Steel temp

Tension Corrections

If a tension other than the Standard tension is used, then a tension correction should be used.

$$C_P = (P - P_s) * L / AE$$

where,

C_P = tension correction per tape length

P = applied tension

P_S = standard tension (4.5 – 5.0 kg (50N))

L = length of tape under consideration

A = tape cross-sectional area

E = average modulus of elasticity of steel tapes (21E5 kg/ sq cm)

- *Hint: 1 kg = 9.807 N*

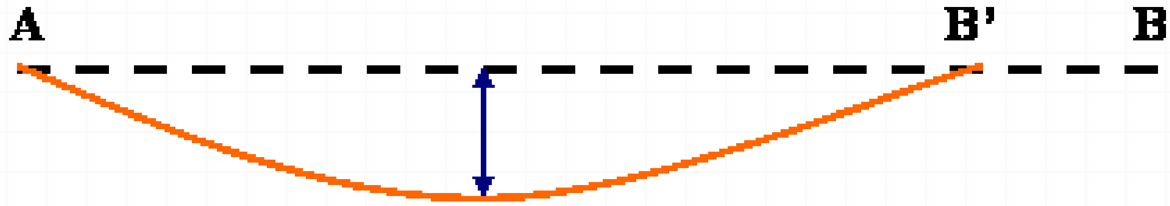
Sag Corrections

If the tape is not fully supported while measurement, then a sag correction should be used.

$$C_S = (- w^2 L^3) / (24\ P^2) = (-W^2\ L) / (24\ p^2)$$

where,

w = weight of tape per unit length
W = weight of tape



Tension Corrections

Example 1. Given Measured distance = 182.716 m

Used tape 30 m

Tape cross sectional area = 0.02 sq. cm

Standard tension force = 50 N

Used tension force = 100 N

Required Corrected measured distance.

Solution : Tension Correction = Cp

= (100 – 50) * 30 / (0.02 * 21E5 * 9.807)

= + 0.0036

Total correction = (182.716/30) * 0.0036 = + 0.022 m

Corrected measured distance = 182.716 + 0.022 = 182.738 m

Sag Corrections

Example 2. Given Measured distance = 42.071 m
Used tape 100 m
Mass of the used part of the tape = 1.63kg
Applied tension force = 100 N
Tape is not fully supported

Required Corrected measured distance.

Solution : Sag Correction = C_S

$$= - (1.63 * 9.807)^2 * 42.07 / 24 * 100^2$$

$$= - 0.045 \text{ m}$$

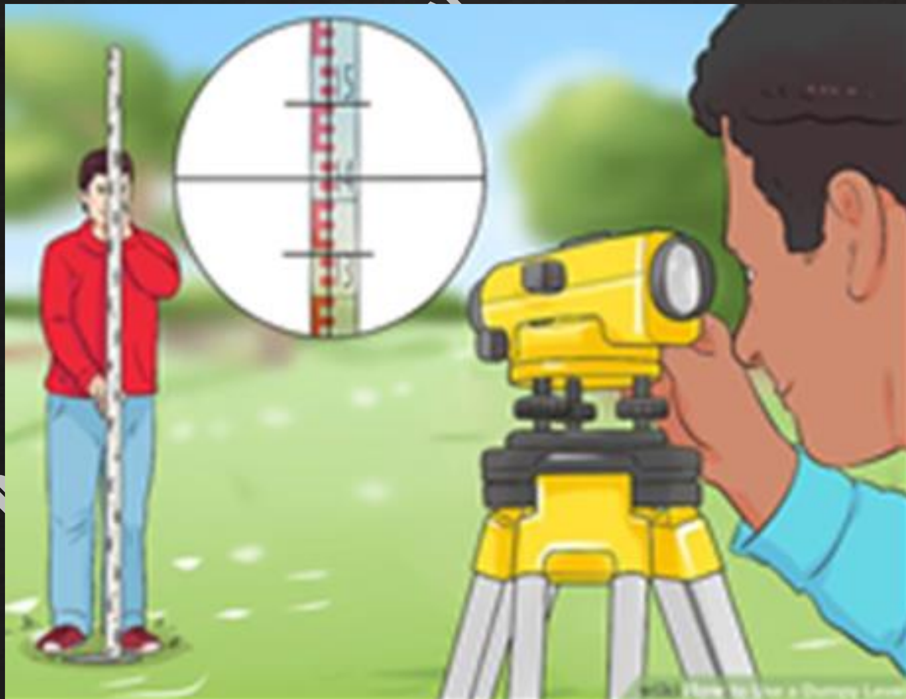
Corrected measured distance = 42.0 - 0.045 = 42.026 m

All Errors can be summed
Tape + Temp + Tension + Sag

Prof. TALEB AL-ROUSAN

Surveying 2104011365

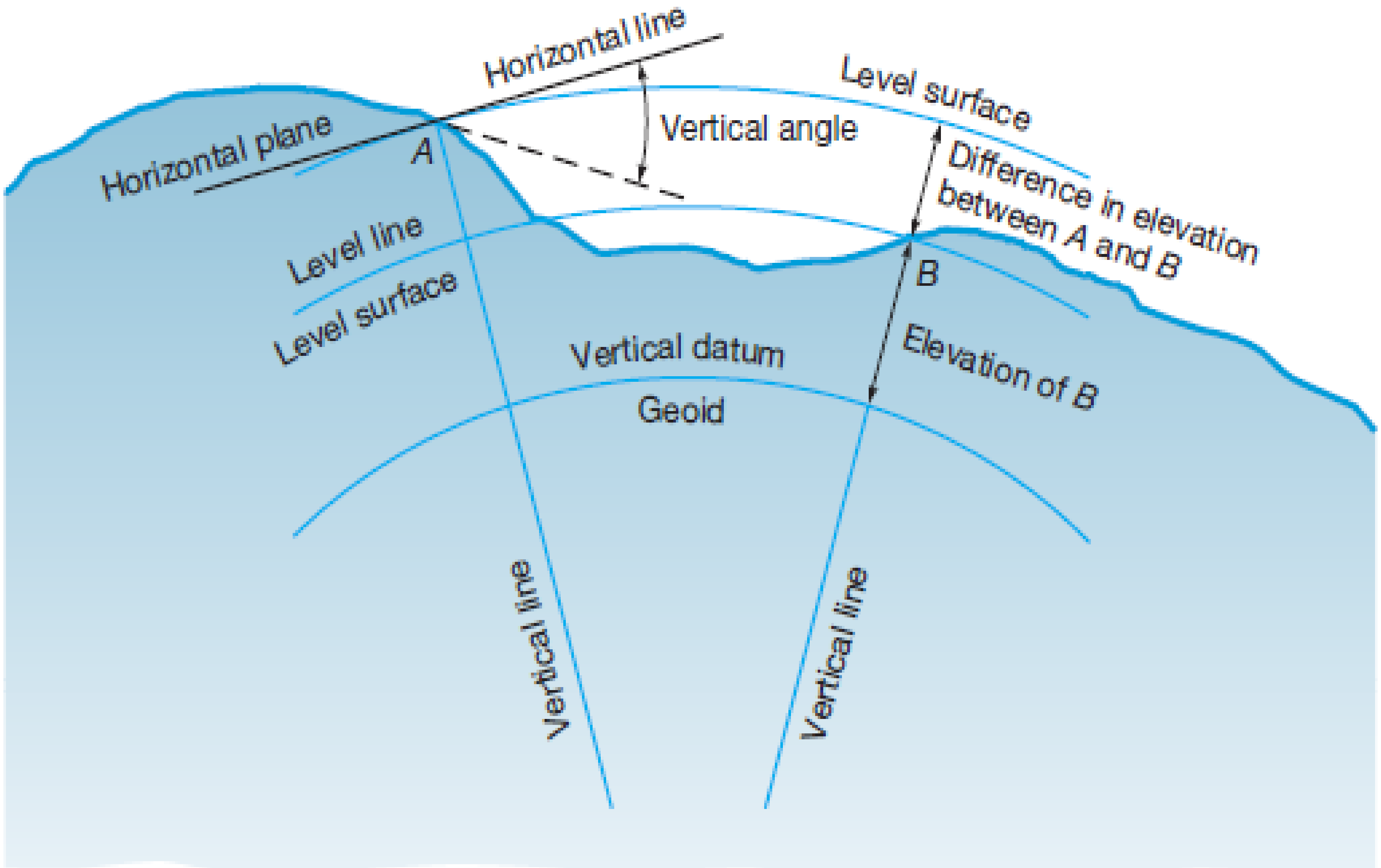
Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University

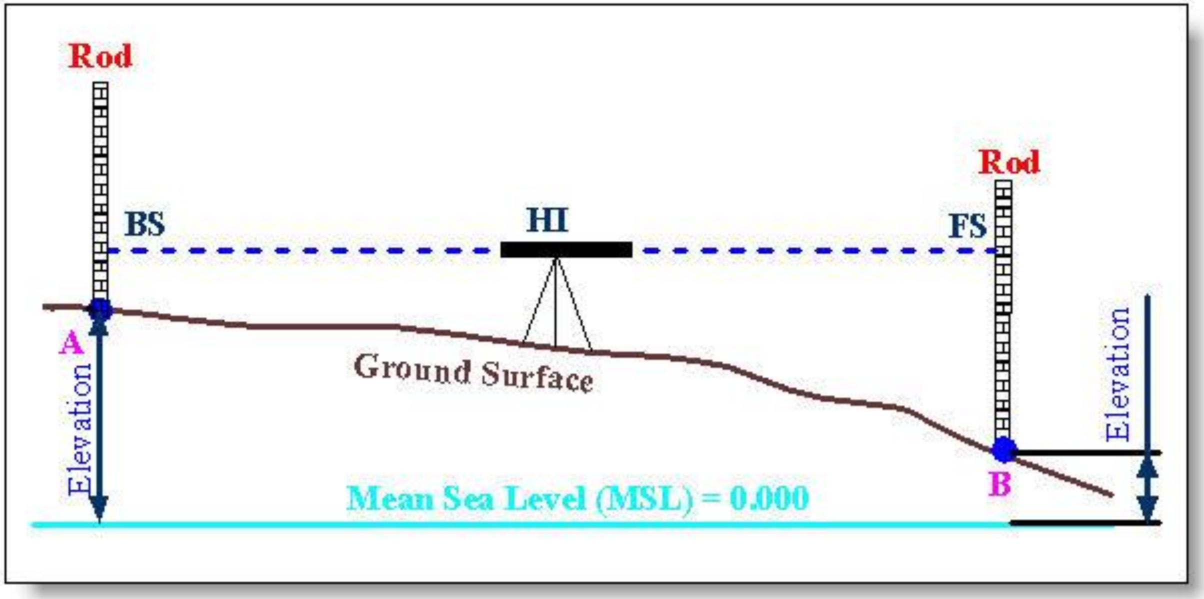


Chapter 3
Leveling

Leveling

- Leveling: Determining differences in elevation between two points
- Elevation: Vertical distance above or below a reference datum (MSL (Mean Sea Level), NAVD88 (North America Vertical Datum 1988)).
- Level surface: curved surface parallel to the mean surface of the earth (i.e. parallel to surface of large body of water) and at every point is perpendicular to the local plumb line (the direction in which gravity acts). Level surfaces are approximately spheroid in shape.
- Level line: a line in a level surface (curved line)
- Horizontal line: a line perpendicular to the vertical line


Horizontal line
Level surface
Vertical angle
Horizontal plane
A
Difference in elevation
between A and B
Level line
B
Level surface
Vertical datum
Elevation of B
Geoid
Vertical line
Vertical line



Leveling Crew

- **The instrument–man:** levels the instrument, takes the readings and calls them loud.
- **Two rod-men:** choose the turning points, so that the distances to and from the instrument are equal and hold the rods vertically on the chosen points using rod levels.
- **The recorder:** takes care of the field book, hears the instrument man, and calls the readings loud to confirm that what he has heard is correct. He records the readings and makes the necessary calculations.

Types of Level Instruments

1. **Dumpy level** *(Press on Link to see details)*

   - **Basic level machine**
   - **Has four leveling screws**

2. **Tilting level**

   - **Advanced leveling technique and usually more accurate**
   - **Has tilting screws**

3. **Automatic level** *(Press on Link to see details)*

   - **Very popular, quick and easy to setup and easy to use and can be obtained for use at any required accuracy.**
   - **Equipped with gravity referenced prism or mirror**

Types of Level Instruments

4. Precise level

- Very precise tilting or automatic levels usually used to establish vertical control

5. Digital level
(Electronic level)

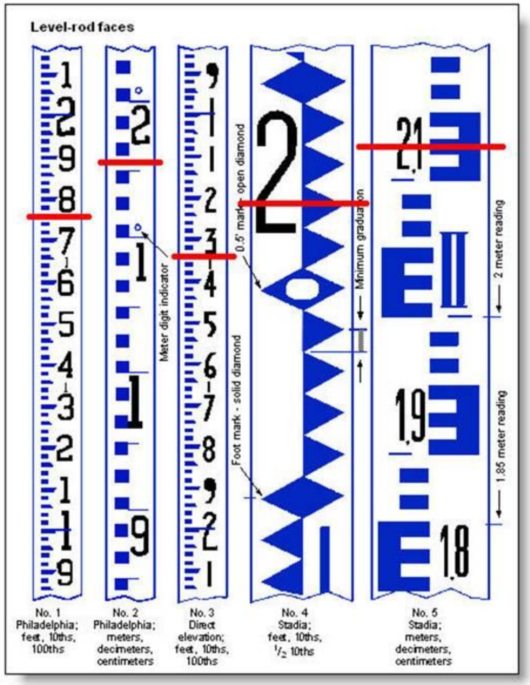
- Automatic level
- Uses coded bars *(Press on Link to see details)*
- Electronic image processing
- Automatic recording of data
- Format of data is similar to total station
- Data can be transferred to computer
- Determine height and distance

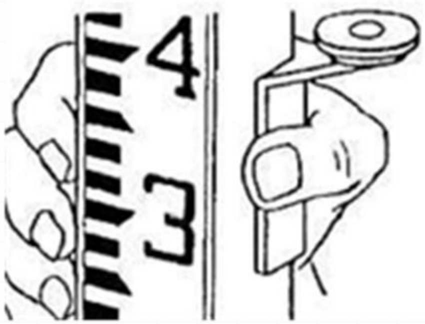
6. Automatic Self-Leveling Rotary Lasers

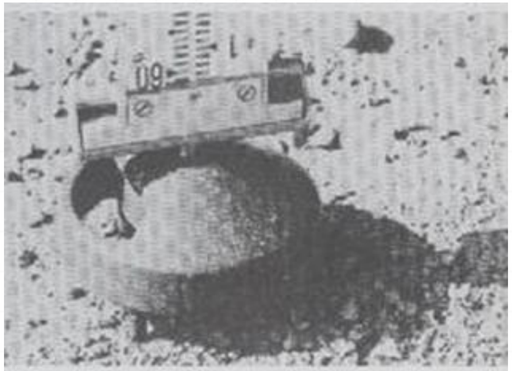
- Transmits laser beams to specify a circular horizontal circle

Leveling Accessories

- Level Rod (staff)
- Rod level
- Rod foot







Types of Leveling Errors:

1. Curvature error, due to earth curvature.

$(R + C)^2 = R^2 + KA^2$

$R^2 + 2RC + C2 = R^2 + KA^2$

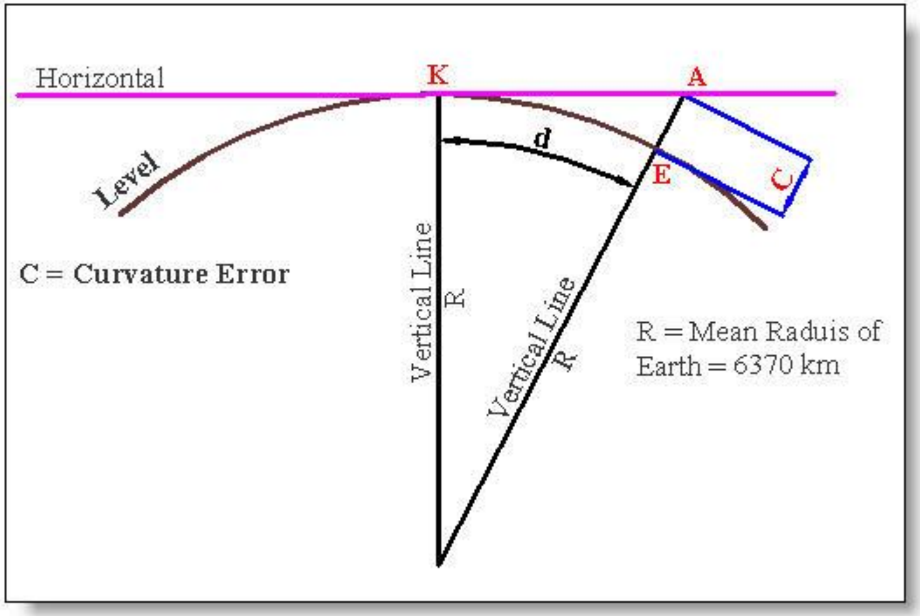
$C(2R + C) = KA^2$

$$C = \frac{KA^2}{2R + C} \approx \frac{KA^2}{2R}$$

$$taking\ R = 6370 \quad km$$

$$C = \frac{KA^2 \times 10^3}{2 \times 6370} \approx 0.0785 KA^2 \quad (m)$$

$$KA : substituted \quad in \quad formulae \quad in \quad km$$



2. Refraction error:

- Sight lines are refracted down by earth atmosphere
- One seventh of curvature error
- In the opposite direction of curvature error

Taking refractor error = 1/7 of curvature error

and $\mathbf{C = 0.0785\ KA^2}$

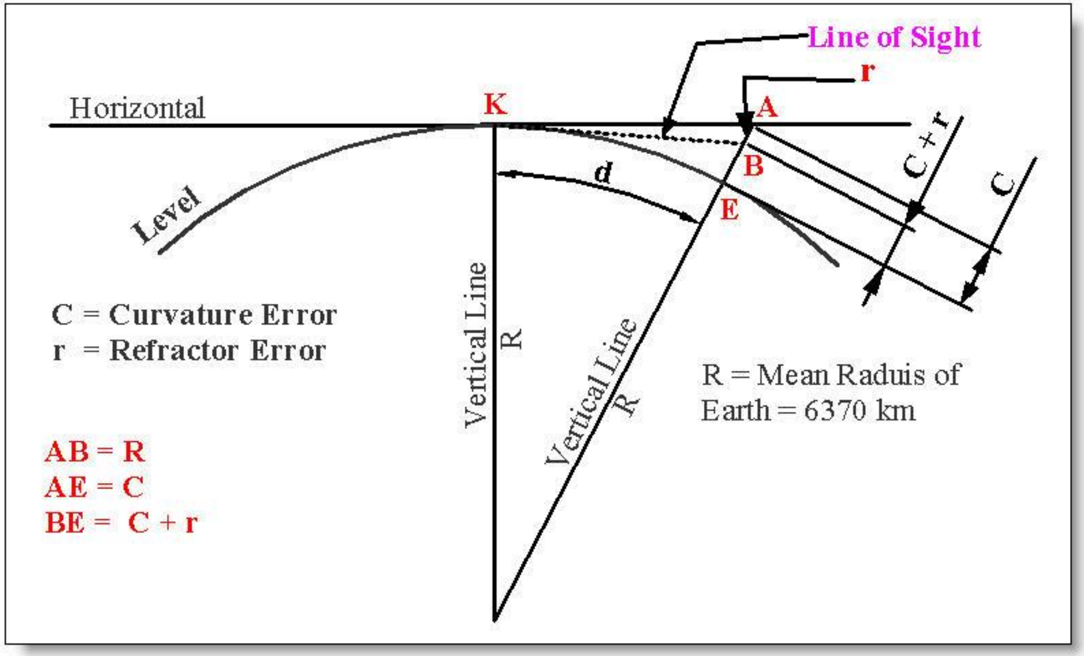
Then C + r = 6/7 C

Shots < 300' (91m): C is negligible

C = .000206' at 100' (30.5 m)

C = .00185' at 200' (61 m)

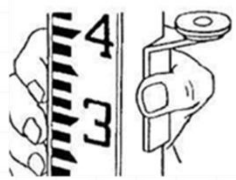
C = .0206' at 1000' (305 m)

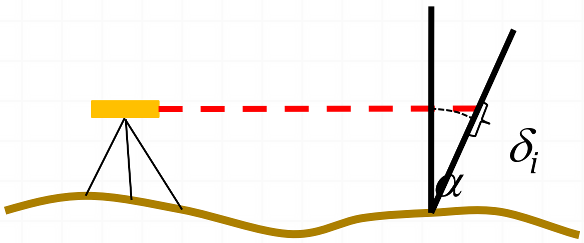


3. Parallax Error:

- Occurs if telescope focus and/or eyepiece lens focus are/is not correct.
- Will lead to cross hair not showing correctly on the rod resulting in errors in the readings.

4. Rod not held vertically straight.





5. Foresights and corresponding back-sights on turning points not equally distant from the instrument.

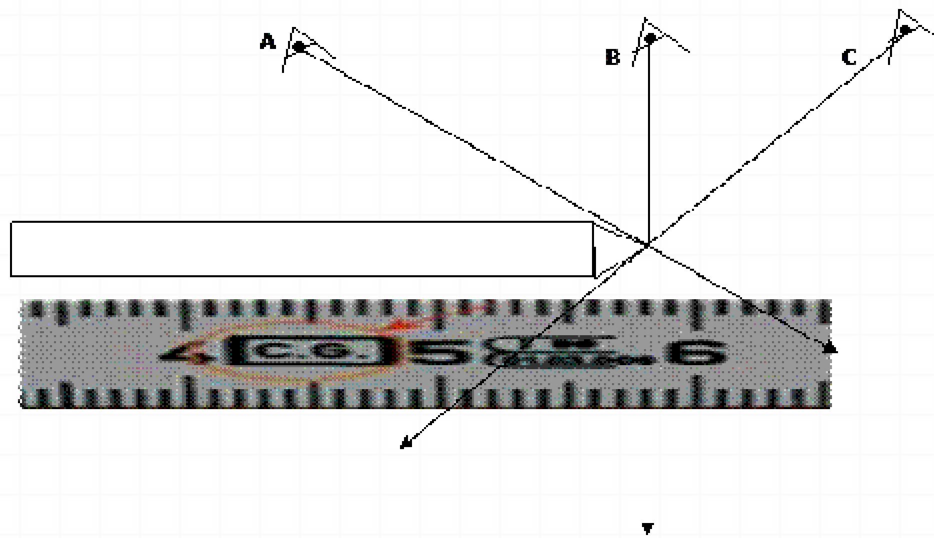
6. Poor turning point selected.

7. Settlement of the tripod when set over soft ground.

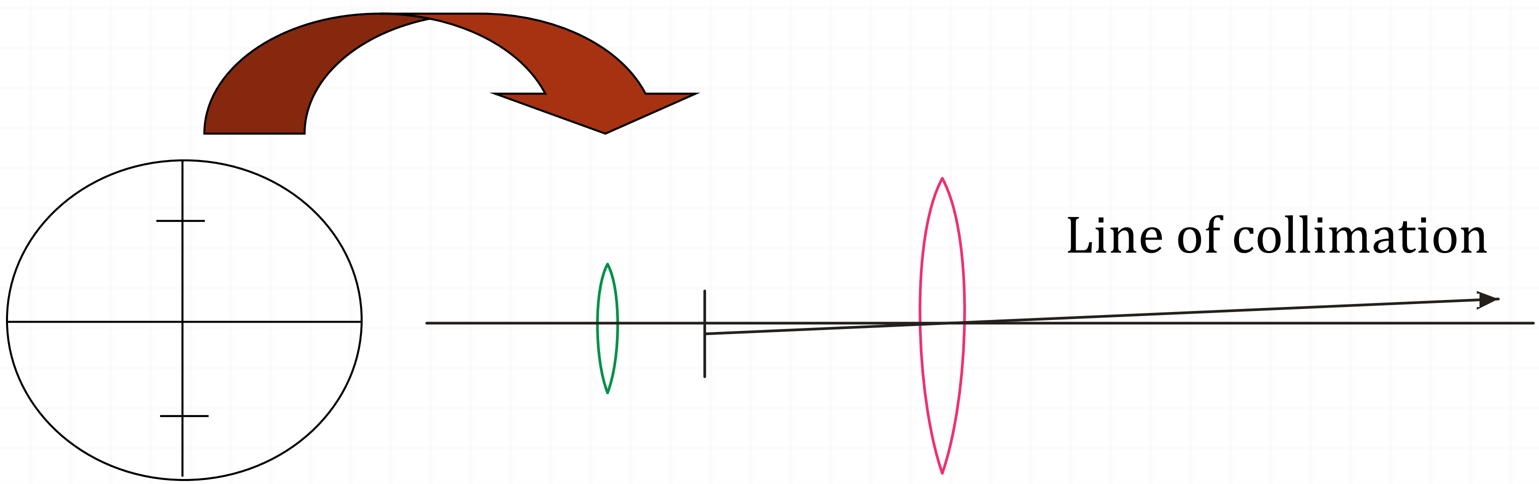
8. Bubble not in middle of tube at instant of sighting.

9. Clump of dirt stuck on the bottom of the rod.

- **Parallax is the change in the apparent position of an object when the position of the observer changes.**
- If you have placed a pencil on a meter rule and you are reading its length then you can place you eye everywhere you want.
- Clearly at these three position we can have the following reading
- Reading at A = 6.2 cm
- Reading at B = 5.8 cm
- Reading at C = 5.5 cm
- What do you think would be the correct reading?
- The correct reading is would be obtained when the eye is placed at B.



The diaphragm (cross-hairs)
To provide visible horizontal and vertical reference lines in the telescope.
With adjustment screws the diaphragm can be moved in the telescope to adjust the line of collimation.



Leveling Operation/ Definitions

- **Benchmark (BM):** A relatively permanent object, natural or artificial, having a marked point whose elevation above or below a reference datum is known or assumed.

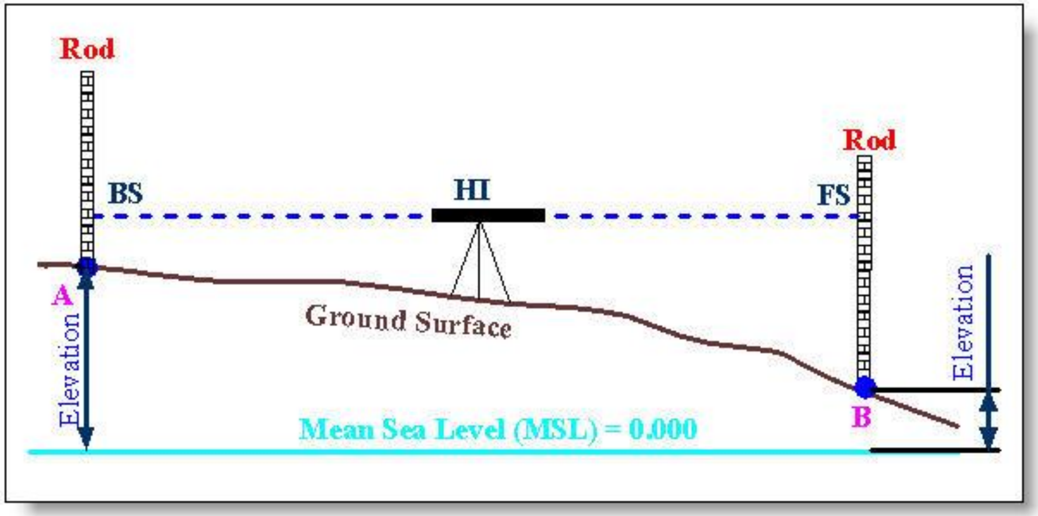
- **Temporary Benchmark (TBM):** is a semi-permanent point of known elevation.

- **Turning point (TP):** is a point temporarily used to transfer an elevation.

- **Backsight (BS):** is a rod reading taken on a point of known elevation to establish the elevation of the instrument line of sight.

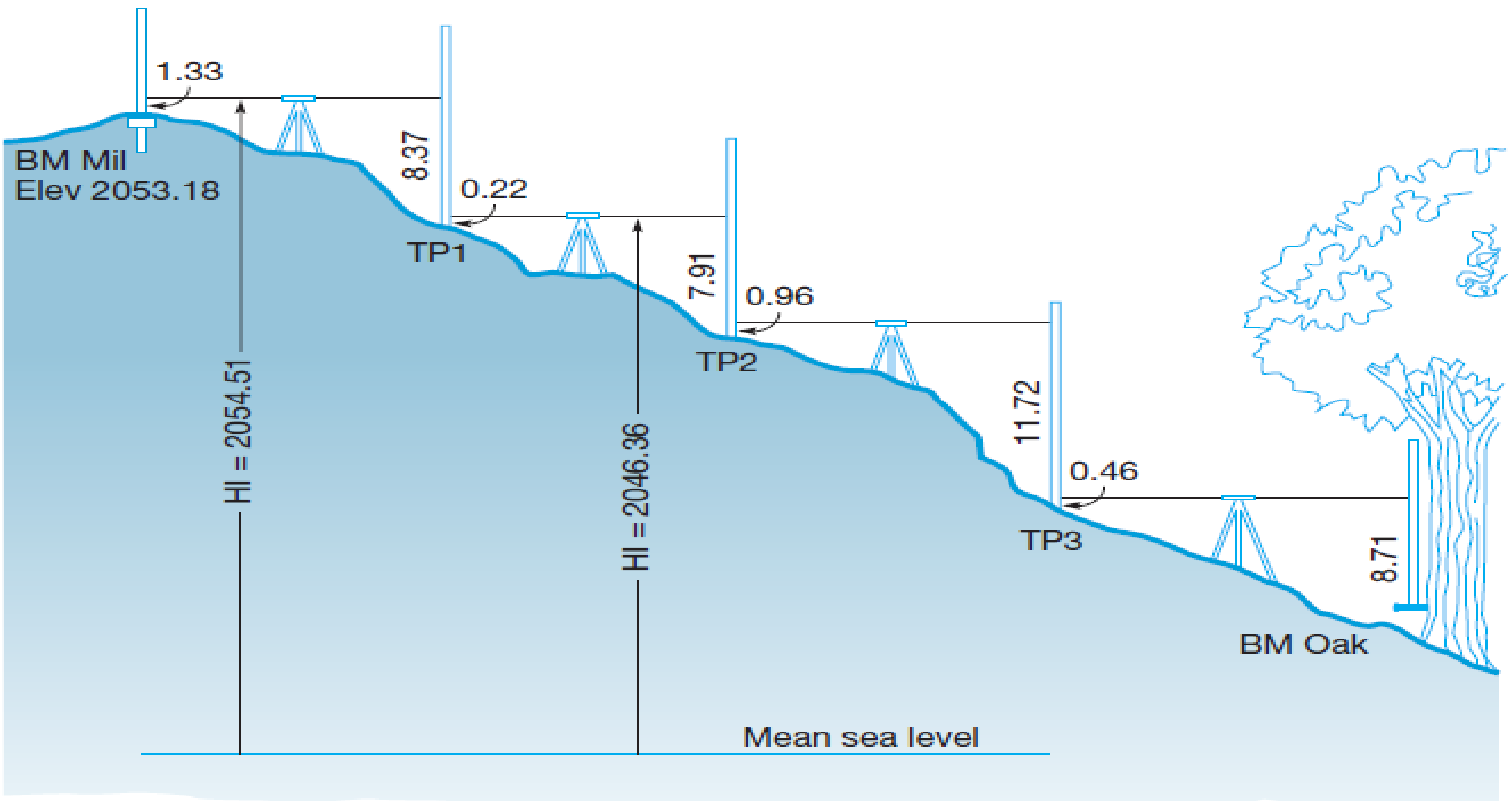
Leveling Operation/ Definitions

- **Height of instrument (HI):** is the elevation of the line of sight through the level (i.e. elev. of BM + BS = HI)
- **Foresight (FS):** is a rod reading taken on a turning point, benchmark, or temporary benchmark to determine its elevation; that is, HI –FS = elev. of TP (or BM or TBM)
- **Intermediate sight (IS):** is a rod reading taken at any other point where the elevation is required; that is HI – IS = elev.



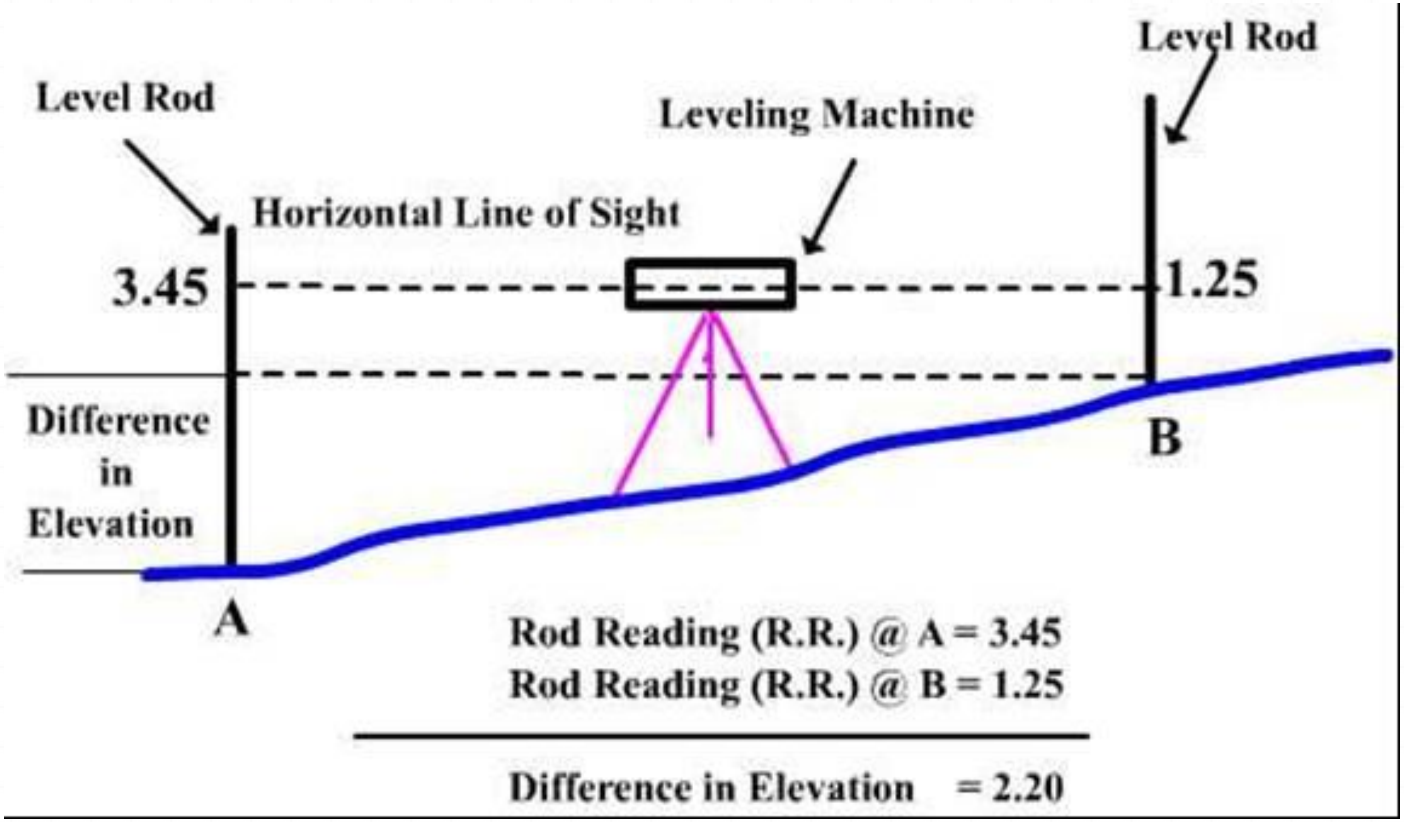
Types of Leveling

1. **Differential Leveling:** is the conventional method of determining the differences in height between survey control .



Procedure & Numerical Example

- From the following figure, you are required to get the difference in elevation between points A & B, and to get elevation of point B if point A is considered a BM and has an elevation of 323.24m.



Procedure & Numerical Example

- Prepare a table in the field notebook for recording the readings according to the method of the instrument height as shown in the following table. Note that all data concerning any point should be recorded on one line in the table.

BS (m)	IS (m)	FS (m)	HI (m)	Elevation (m)	Remarks
3.45			326.69	323.24	Point A
		1.25		325.44	Point B
3.45		1.25			Sum
		2.20		2.20	Difference

Procedure & Numerical Example

- 2. Set and level the instrument at approximately mid distance between points A & B.

- 3. Rodman I should hold rod I vertically on bench mark (A). The elevation of the bench mark (A) (323.24) should be entered in the table in the column headed "Elevation". The instrument-man takes the rod reading at (A) (3.45 m) and calls it loud. The recorder repeats it loud while he is recording it in the column of back sight (BS).

- 4. Calculate the height of instrument above datum by adding the BS reading to the given elevation:

  HI = Existing Elevation + BS

  Instrument height = 323.24 + 3.45 = 326.69 m.

- **5.** Hold rod vertically on point B, take the reading (1.25 m) and record it in the column of foresight (FS). Calculate its elevation by subtracting the FS reading from the instrument height:

  New Elevation = HI - FS

  Elevation of point B = 326.69 - 1.25 = 325.44 m.

Record this value in the column under "elevation".

Procedure & Numerical Example

- 7. Add all backsights (BS) and all foresights (FS) and calculate the difference between both sums.

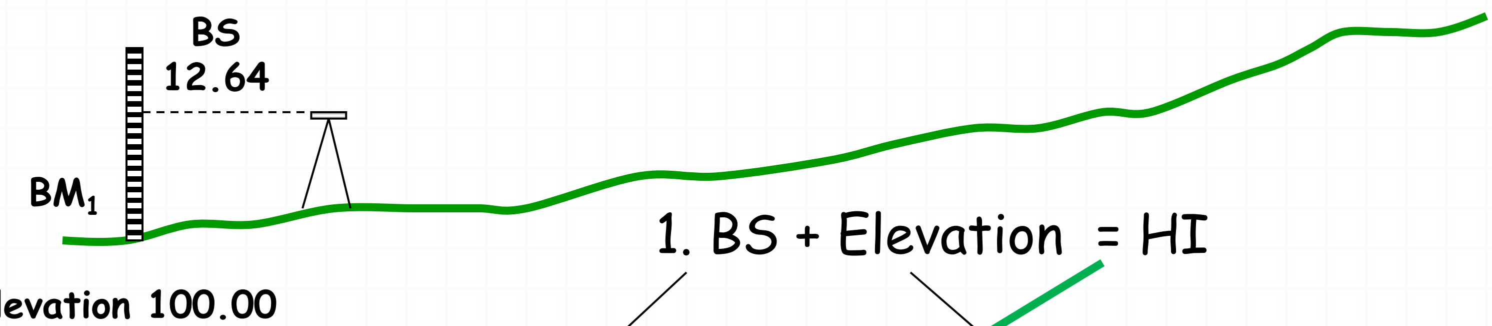
Sum Bs – Sum Fs = 3.45 – 1.25 = 2.20 m

- 8. Compare this result with the difference in elevation between the last point and the first point (the bench mark).

Elevation of "B"- Elevation of "A" = 325.44 - 323.24 = 2.20 m

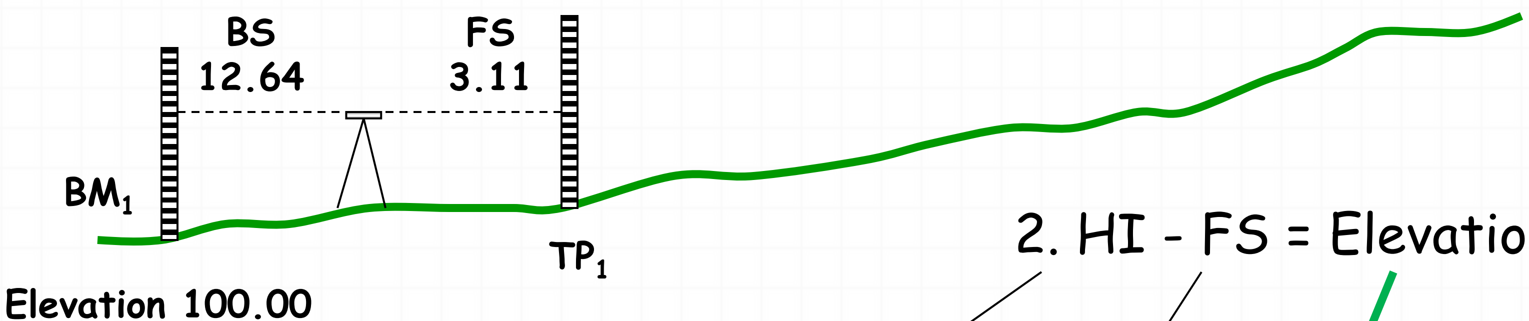
Both values must be exactly the same

- 9. The mathematical check mentioned in step 8 should be carried out whenever a field-book page is completed and at the end of every leveling. ***It ensures that no errors in addition or subtraction have taken place. This is by no means a check for the correctness of the leveling.***
- **In normal practice, to check accuracy of measurements, the leveling is checked by closing it to either a point of known elevation or to the point of beginning by back leveling.**



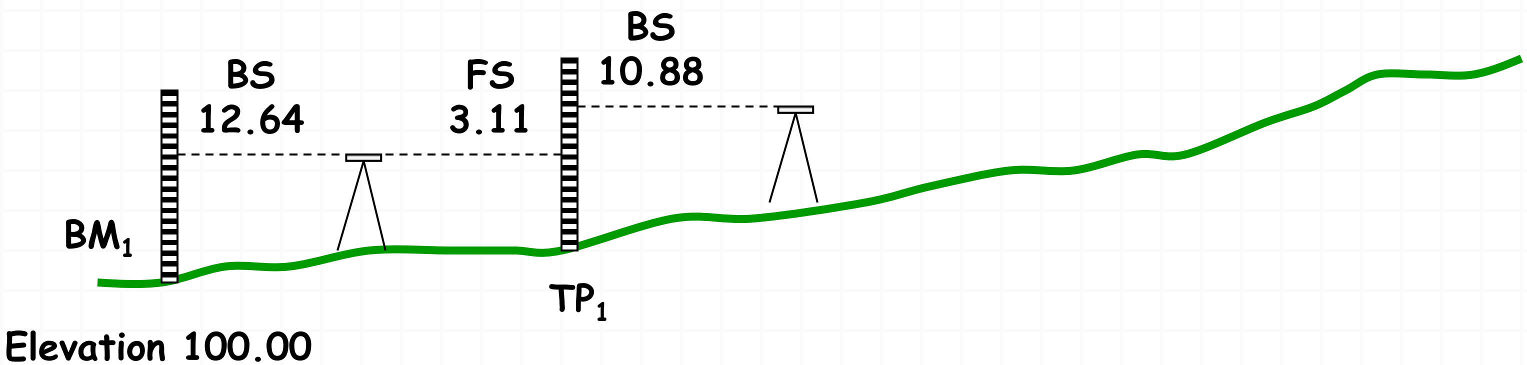
1. BS + Elevation = HI

Point	BS	HI	FS	Elevation
BM_1	12.64	112.64		100.00

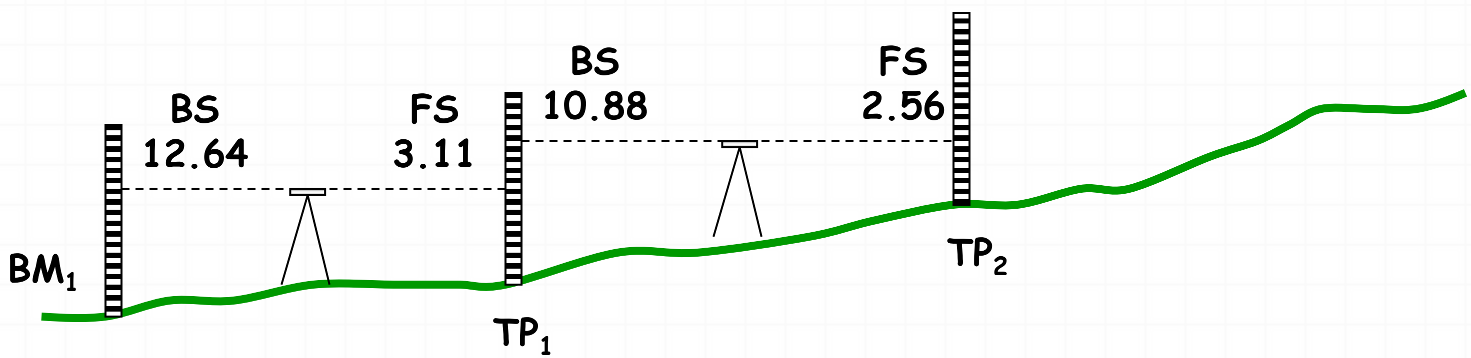


2. HI - FS = Elevation

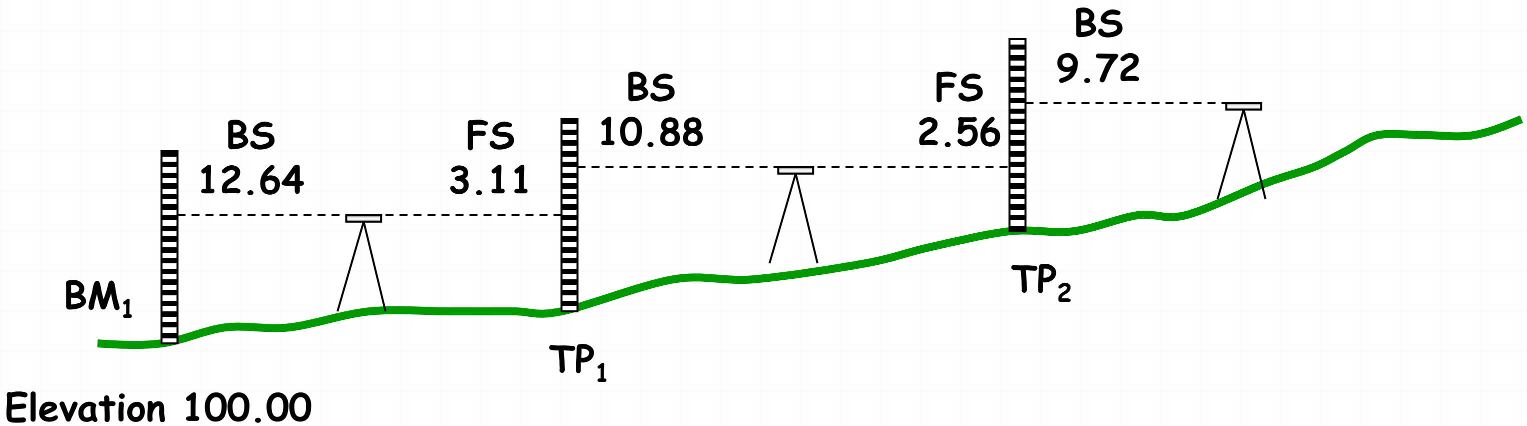
Point	BS	HI	FS	Elevation
BM_1	12.64	112.64		100.00
TP_1			3.11	109.53



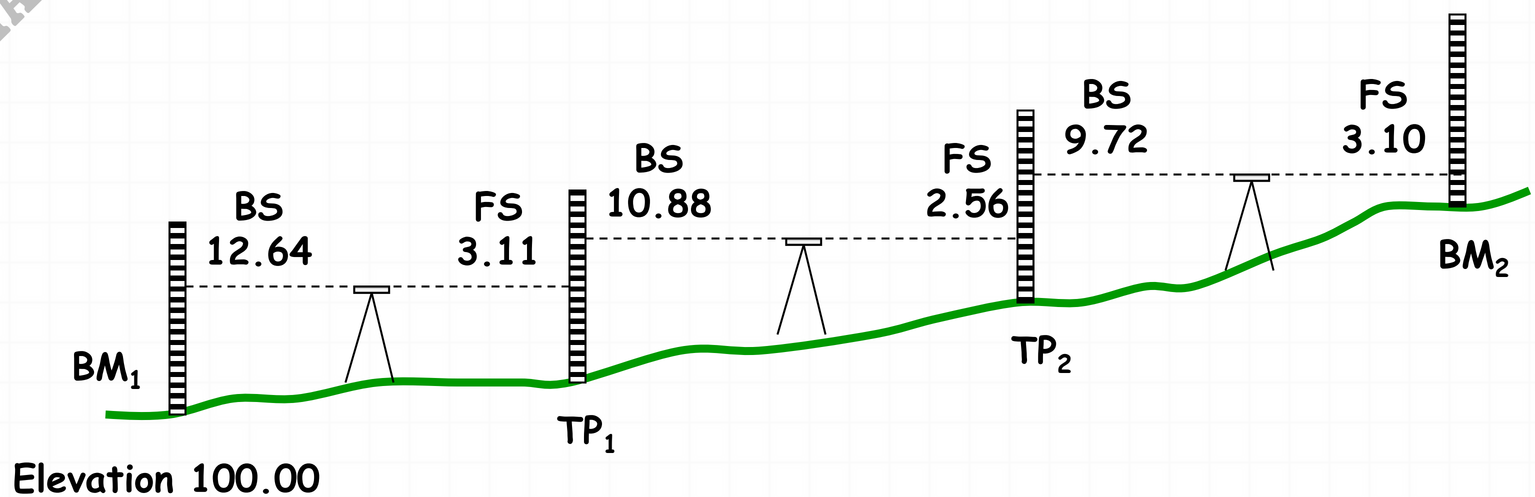
Point	BS	HI	FS	Elevation
BM_1	12.64	112.64		100.00
TP_1	10.88	120.41	3.11	109.53



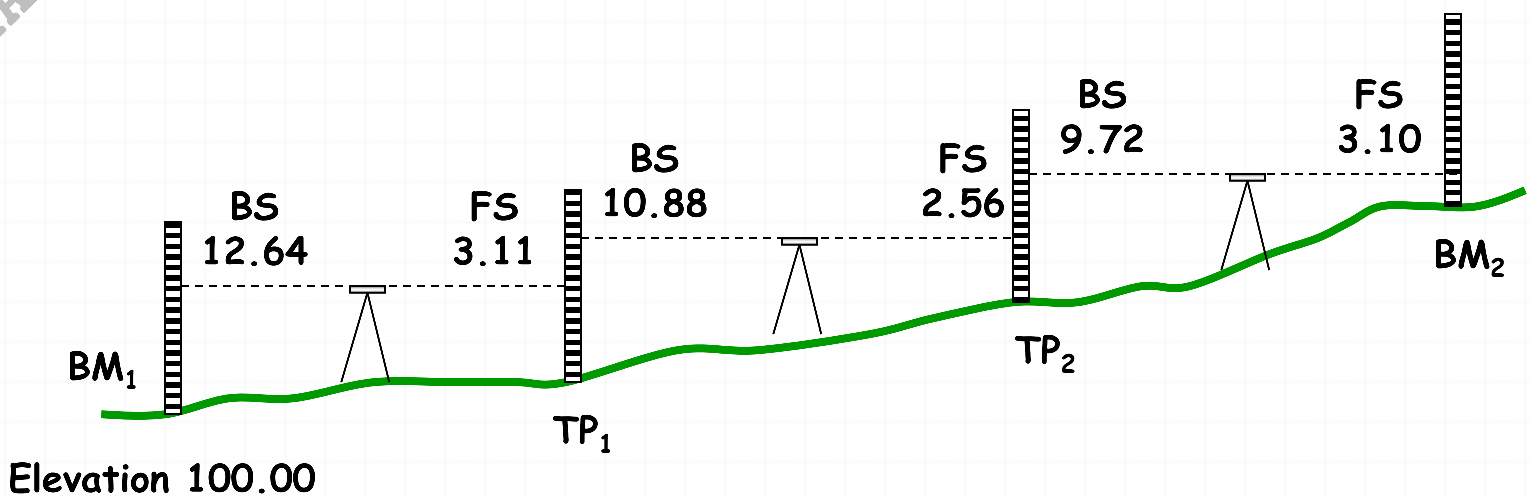
Point	BS	HI	FS	Elevation
BM_1	12.64	112.64		100.00
TP_1	10.88	120.41	3.11	109.53
TP_2			2.56	117.85



Point	BS	HI	FS	Elevation
BM_1	12.64	112.64		100.00
TP_1	10.88	120.41	3.11	109.53
TP_2	9.72	127.57	2.56	117.85



Point	BS	HI	FS	Elevation
BM_1	12.64	112.64		100.00
TP_1	10.88	120.41	3.11	109.53
TP_2	9.72	127.57	2.56	117.85
BM_2			3.10	124.47



Point	BS	HI	FS	Elevation
BM_1	12.64	112.64		100.00
TP_1	10.88	120.41	3.11	109.53
TP_2	9.72	127.57	2.56	117.85
BM_2			3.10	124.47

3. Change in elevation- summation of the backsight and the foresight then subtract

Point	BS	HI	FS	Elevation
BM_1	12.64	112.64		100.00
TP_1	10.88	120.41	3.11	109.53
TP_2	9.72	127.57	2.56	117.85
BM_2			3.10	124.47

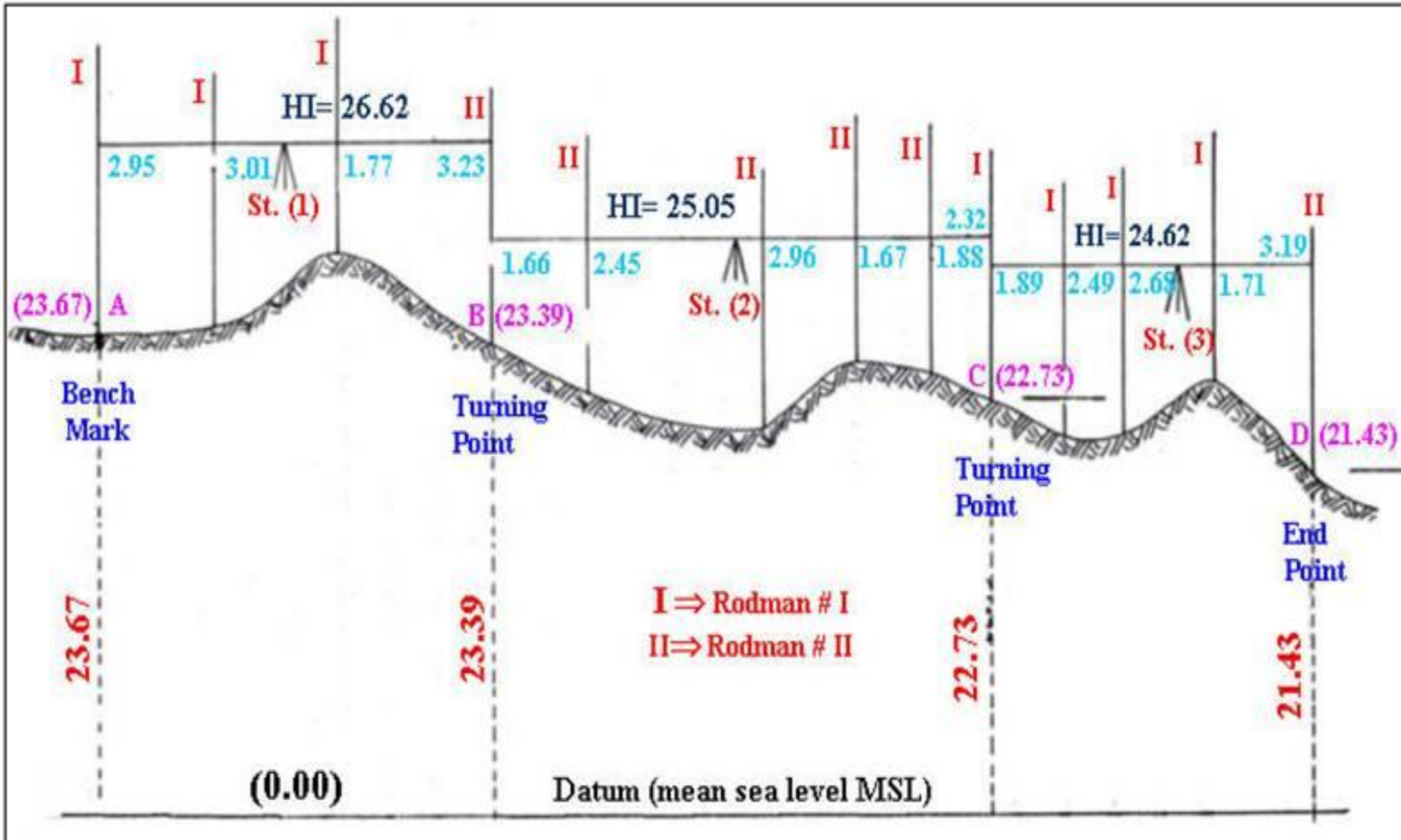
+33.24 -8.77

Change in elevation = **33.24 -8.77 =24.47**

4. The initial ***backsight (BS)*** is taken to a point of known elevation
5. The backsight reading is added to the elevation of the known point to compute the ***height of the instrument (HI)***
6. The level may be moved to a temporary point called a ***turning point (TP)***
7. The elevation of a point is the ***height of the instrument (HI)*** minus the ***foresight (FS)***

Numerical Example

From the following figure, you are required to get the difference in elevation between points A & D, and to get elevation of point D if point A is considered a BM and has an elevation of 23.67m. Points A & D are far apart, therefore two turning points are required. In addition, you are required to find elevations of the intermediate points.

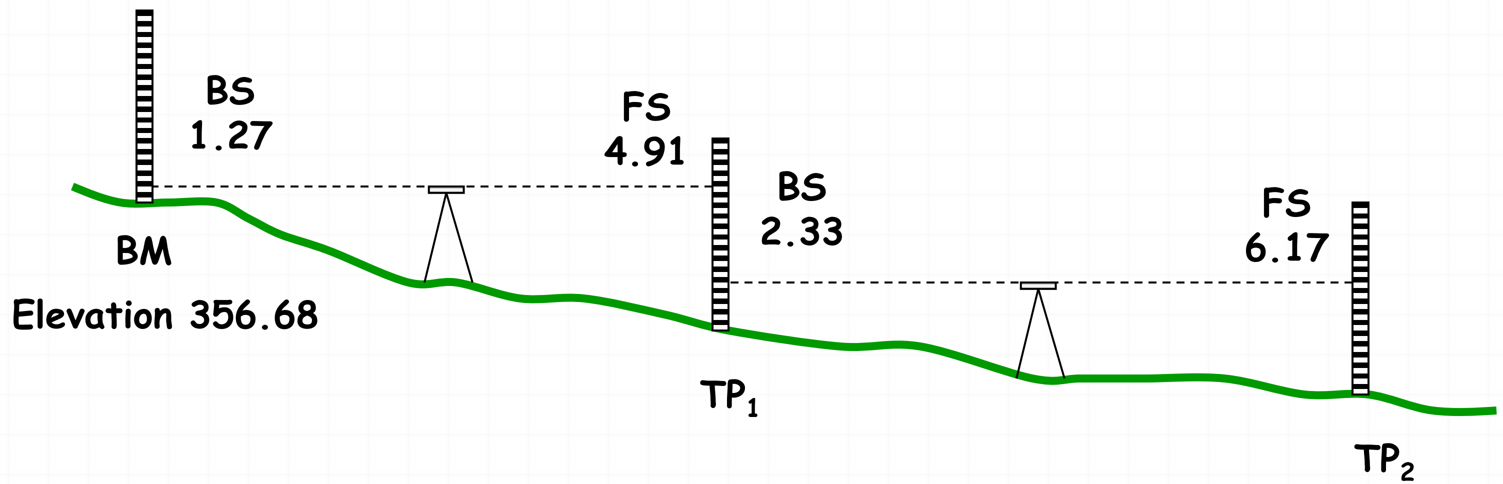


HI= 26.62
HI= 25.05
HI= 24.62
2.95
3.01
1.77
3.23
1.66
2.45
2.96
1.67
2.32
1.88
1.89
2.49
2.68
1.71
3.19
St. (1)
St. (2)
St. (3)
(23.67) A
B (23.39)
C (22.73)
D (21.43)
Bench Mark
Turning Point
Turning Point
End Point
23.67
23.39
22.73
21.43
I ⇒ Rodman # I
II⇒ Rodman # II
(0.00)
Datum (mean sea level MSL)

BS (m)	IS (m)	FS (m)	HI (m)	Elevation (m)	Remarks
2.95			26.62	23.67	Point A (BM)
	3.01			23.61	
	1.77			24.85	
1.66		3.23	25.05	23.39	Point B
	2.45			22.60	
	2.96			22.09	
	1.67			23.38	
	1.88			23.17	
1.89		2.32	24.62	22.73	Point C
	2.49			22.13	
	2.68			21.94	
	1.71			22.91	
		3.19		21.43	Point D
6.50		8.74			Sum
	-2.24			-2.24	Difference

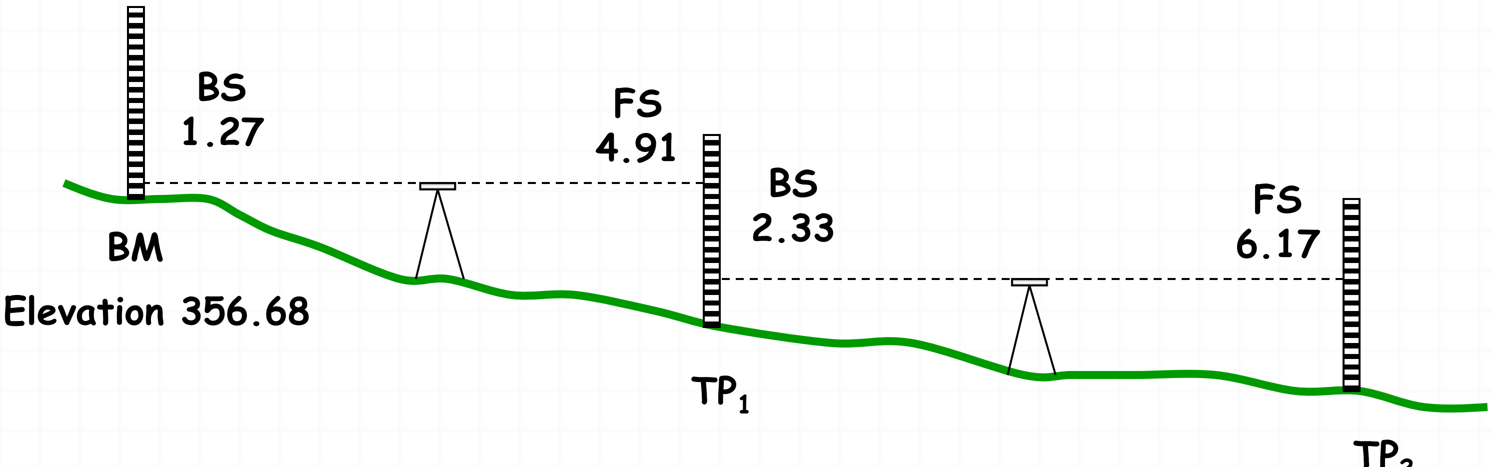
Example:

- Prepare a set of level notes for the survey illustrated below. What are the elevations of points TP_1 and TP_2?



Differential Leveling

Computation of Elevations



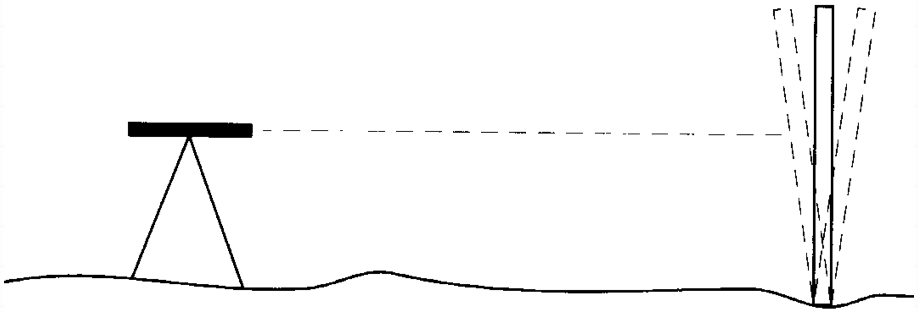
Point	BS	HI	FS	Elevation
BM_1	1.27	357.95		356.68
TP_1	2.33	355.37	4.91	353.04
TP_2			6.17	349.20
	+3.60		-11.08	-7.48

Differential Leveling

Common Mistakes

1. Misreading the rod - *reading 3.54 instead of 3.45*
2. Moving the turning point - *use a well-defined TP*
3. Field note mistakes - *work within your group to check you records*
4. Mistakes with extended rod - *make sure the leveling rod is fully extended*

Common Mistakes



5. Level rod not vertical
6. Settling of leveling rod
7. Leveling rod not fully extended or incorrect length
8. Level instrument not level
9. Instrument out of adjustment
10. Environment - wind and heat

Suggestions for Good Leveling

1. Anchor tripod legs firmly
2. Check the bubble level before and after each reading
3. Take as little time as possible between BS and FS
4. Try to keep the distance to the BS and the FS equal
5. Provide the rod person with a level for the rod

Closed Leveling

In normal practice, to check accuracy of measurements, the leveling is checked by closing it to either a point of known elevation (might be a BM) or to the point of beginning by back leveling.

The final elevation **should agree with the starting elevation** if returning to the initial benchmark. The amount by which they differ is the **loop misclosure**.

If closure is made to another benchmark, **the section misclosure is the difference between the closing benchmark's given elevation and its elevation obtained after leveling through the section**.

Leveling error/ misclosure

The Federal Geodetic Control Subcommittee (FGCS) recommends the following formula to compute allowable misclosures:

$$C = m\sqrt{K}$$

Where:
C is the allowable loop or section misclosure, in millimeters;
m: is a constant; and
K: the total length leveled, in kilometers.

For "loops" (circuits that begin and end on the same benchmark), K is the total perimeter distance, and the FGCS specifies constants of **4**, **5**, **6**, **8**, and **12 mm** for the five classes of leveling, designated, respectively, as **(1) first-order class I**, **(2) first-order class II**, **(3) second-order class I**, **(4) second-order class II**, and **(5) third-order**.

Leveling error/ misclosure

- If the allowable misclosure is exceeded, one or more additional runs must be made (survey operation has to be repeated).
- When acceptable misclosure is achieved, final elevations are obtained by making an adjustment.
- The error is distributed according to traveled distance between the traverse points. Therefore, correction to the observed elevation of any point included in the survey is:

$$\text{Elevation Correction} = \frac{\textit{Distance from previous point}}{\textit{Total travelled distance}} \cdot \times \textit{Error}$$

Example:

A differential leveling loop is run from an established BM A to a point 3.2 km away and back, with a misclosure of 17 mm. What order leveling does this represent?

$$m = \frac{C}{\sqrt{K}} = \frac{17}{\sqrt{3.2*2}} = 6.7$$

This leveling meets the **allowable 8-mm tolerance level for second-order class II** work, but does not quite meet the 6-mm level for second-order class I.

Since distance leveled is proportional to number of instrument setups, the misclosure criteria can be specified using that variable.

ADJUSTMENTS OF SIMPLE LEVEL CIRCUITS

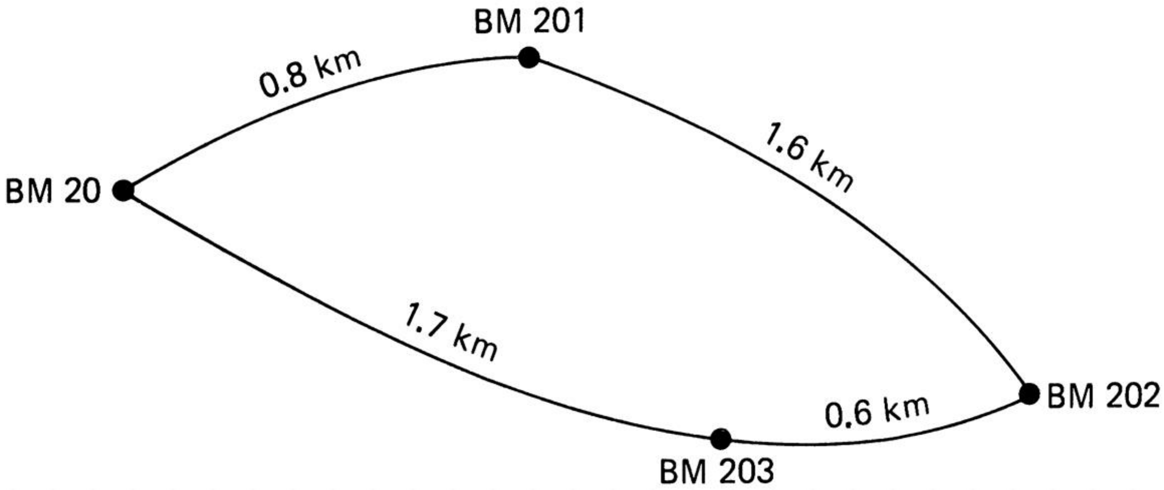


Table 3.4 LEVEL LOOP ADJUSTMENTS

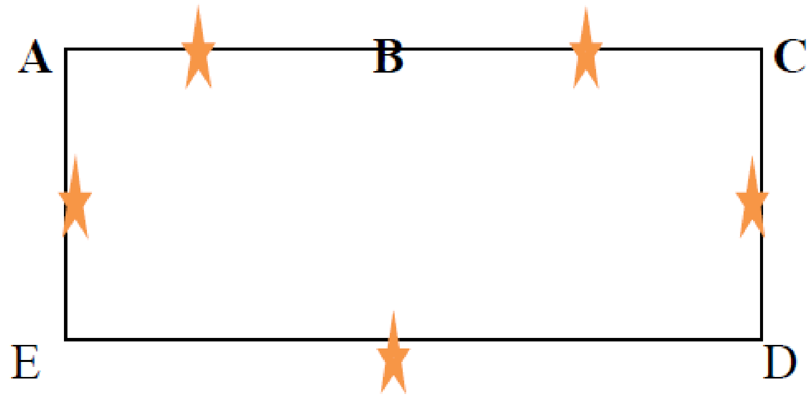
BM	Loop Distance: Cumulative (km)	Field Elevation	Correction: $\frac{\text{cumulative distance}}{\text{total distance}} \times E^*$	Adjusted Elevation
20		186.273 (fixed)		186.273
201	0.8	184.242	$+0.8/4.7 \times 0.015 = +0.003 =$	184.245
202	2.4	182.297	$+2.4/4.7 \times 0.015 = +0.008 =$	182.305
203	3.0	184.227	$+3.0/4.7 \times 0.015 = +0.010 =$	184.237
20	4.7	186.258	$+4.7/4.7 \times 0.015 = +0.015 =$	186.273

$^*E = 186.273 - 186.258 = -0.015$ m.

If the distances are not known then the correction can be distributed according to the number of instrument positions

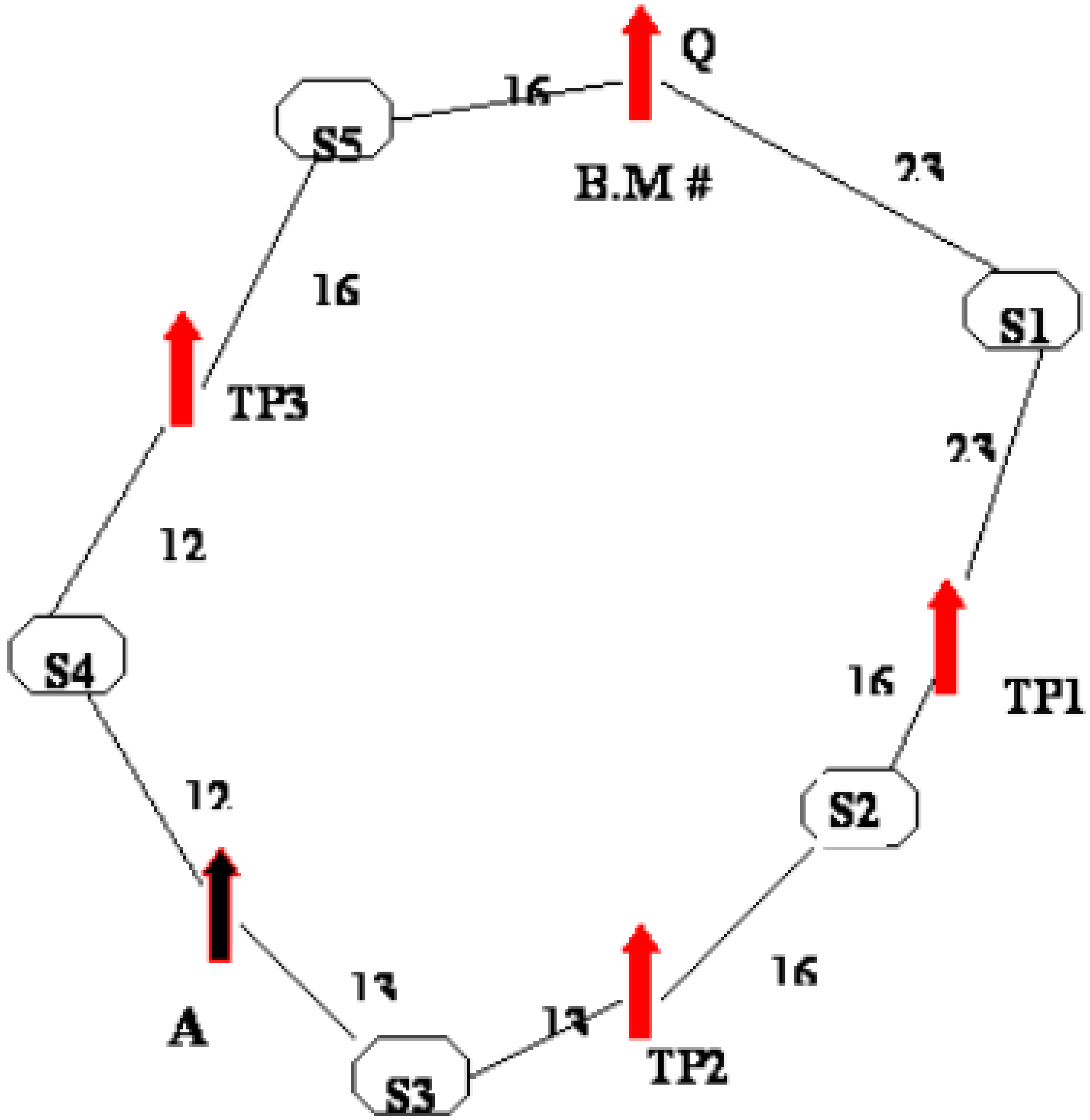
Example:

point	measured elevation	number of set up to the point	correction	corrected elevation
A	100.000	0	-0.02*(0/5)= 0.000	100.000
B	102.458	1	-0.02*(1/5)= -0.004	102.454
C	103.539	2	-0.02*(2/5)= -0.008	103.531
D	102.553	3	-0.02*(3/5)= -0.012	102.541
E	101.389	4	-0.02*(4/5)= -0.016	101.373
A	100.02	5	-0.02*(5/5)= -0.020	100.000



Example:

For the following closed leveling Complete the table



Example:

For the following closed leveling Complete the table

BS	FS	HI	Elevation	Cum Distance	Correction	Corrected Elev.	Remarks
0.225			49.521	0		49.521	BM
0.445	1.995			46			P1
1.253	2.415			78			P2
2.425	1.265			104			A
2.324	0.533			128			P3
	0.473			160			BM

	BS	FS	HI	Elevation	Cum Distance	Correction	Corrected Elev.	Remarks
	0.225		49.746	49.521	0	0.000	49.521	BM
	0.445	1.995	48.196	47.751	46	0.003	47.754	P1
	1.253	2.415	47.034	45.781	78	0.004	45.785	P2
	2.425	1.265	48.194	45.769	104	0.006	45.775	A
	2.324	0.533	49.985	47.661	128	0.007	47.668	P3
		0.473		49.512	160	0.009	49.521	BM
SUM	6.672	6.681						
Diff.		-0.009		-0.009				

Correction for P1 = 0.009 *(46/160) = 0.003

DIFFERENTIAL LEVELS

Sta.	+ B.S.	H.I.	− F.S.	Elev.	Adj. Elev.
BM Mil.	1.33			2053.18	2053.18
		2054.51		(−0.004)	
TP1	0.22		8.37	2046.14	2046.14
		2046.36		(−0.008)	
TP2	0.96		~~8.91~~ 7.91	203[illegible]	2038.44
		2039.41		(−0.0[illegible])	
TP3	0.46		11.72	2027.69	2027.68
		2028.15		(−0.016)	
BM Oak	11.95		8.71	2019.44	2019.42
		2031.39		(−0.022)	
TP4	12.55		2.61	2028.78	2028.76
		2041.33		(−0.026)	
TP5	12.77		0.68	2040.65	2040.62
		2053.42		(−0.030)	
BM Mil.			0.21	2053.21	2053.18
Σ =	+40.24	Σ =	−40.21		

Page Check:

2053.18
+ 40.24
2093.42
− 40.21
2053.21 Check

GRAND LAKES UNIV. CAMPUS

BM Mil. to BM Oak

BM Mil. on GLU Campus	29 Sept. 2000
SW of Engineering Bldg.	Clear, Warm 70° F
9.4 ft. north of sidewalk	T.E. Henderson N
to instrument room and	J.F. King ⊕
1.6 ft. from Bldg. Bronze	D.R. Moore ⊼
disk in concrete flush	Lietz Level #6
with ground, stamped "Mil"	

BM Oak is a temporary project bench mark located at corner of Cherry and Pine Sts., 14 ft. West of computer laboratory. Twenty penny spike in 18″ Oak tree, 1 ft. above ground.

Loop Misclosure = 2053.21 − 2053.18 = 0.03

Permissible Misclosure $= 0.02\sqrt{n} = 0.02\sqrt{7}$ $= 0.05$ ft.

Adjustment $= \frac{0.03}{7} = 0.004'$ per H.I.

2. Profile Leveling:

- Profile leveling consists simply of differential leveling with the **addition of intermediate minus sights (foresights) taken at required points along the reference line**.
- Whether the stationing is in feet or meters, intermediate sights are usually taken at all full stations.
- If stationing is in feet and the survey area is in rugged terrain or in an urban area, the specifications may require that readings also be taken at half- or even quarter-stations.
- If stationing is in meters, depending on conditions, intermediate sights may be taken at 40-, 30-, 20-, or 10-m increments.
- In any case, sights are also taken at high and low points along the alignment, as well as at changes in slope.
- Intermediate sights should always be taken on “critical” points such as railroad tracks, highway centerlines, gutters, and drainage ditches.

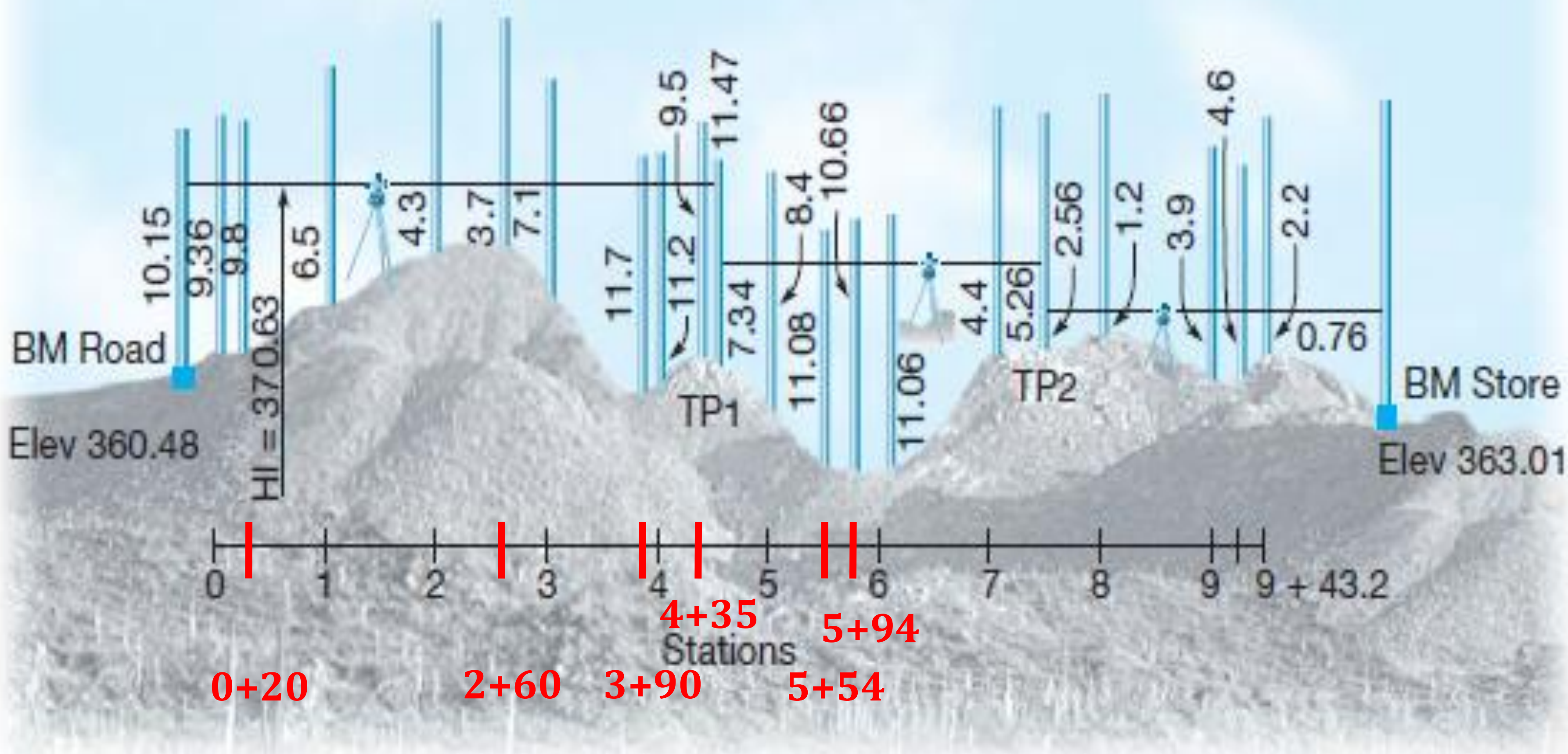
2. Profile Leveling:

As in differential leveling, the page check should be made for each left-hand sheet. However in profile leveling, intermediate minus sights play no part in this computation.

The page check is made by adding the algebraic sum of the column of plus sights and the column of minus sights to the beginning elevation. This should equal the last elevation tabulated on the page for either a turning point or the ending benchmark if that is the case

In the adjustment process, **HIs** are adjusted, because they will affect computed profile elevations. The adjustment is made progressively in proportion to the total number of **HIs** in the circuit.

After adjusting the HIs, profile elevations are computed by subtracting intermediate minus sights from their corresponding *adjusted HIs.*



BM Road
Elev 360.48
HI = 370.63
10.15
9.36
9.8
6.5
4.3
3.7
7.1
11.7
11.2
9.5
11.47
7.34
TP1
8.4
11.08
10.66
11.06
4.4
5.26
TP2
2.56
1.2
3.9
4.6
2.2
0.76
BM Store
Elev 363.01
0
1
2
3
4
5
6
7
8
9
9 + 43.2
Stations
0+20
2+60
3+90
4+35
5+54
5+94

PROFILE LEVELS

Station	+ Sight	HI	− Sight	Int. Sight	Unadj. e	Adj. elev.
BM Road	10.15	(370.62) 370.63			360.48	
0+00				9.36	361.27	361.26
0+20				9.8	361.83	360.82
1+00				6.5	364.13	364.12
2+00				4.3	366.33	366.32
2+60				3.7	366.93	366.92
3+00				7.1	363.53	363.52
3+90				11.7	358.93	358.92
4+00				11.2	359.43	359.42
4+35		(366.48)		9.5	361.13	361.12
TP1	7.34	366.50	11.47		359.16	359.15
5+00				8.4	358.1	358.08
5+54				11.08	355.4	355.40
5+74				10.66	355.8	355.82
5+94				11.06	355.4	355.42
6+00		(363.77)		10.5	355.9	356.0
7+00				4.4	8 362.1	362.08
TP2	2.56	363.8	5.26		361.24	361.22
8+00				1.2	362.6	362.57
9+00				3.9	359.9	359.87
9+25.2				3.4	360.4	360.37
9+25.3				4.6	359.2	359.17
9+43.2				2.2	361.6	361.57
BM Store			0.76		363.04	363.01
Σ	20.05		17.49		(363.01)	

BM ROAD to BM STORE

BM Road 3 miles SW of Mpls. 200 yrds. N of Pine St. over pass 40ft. E of ℄ Hwy. 169 Top of RW conc post No.268.	SW Minneapolis on Hwy 169 / 6 Oct. 2000
℄ Hwy. 169, painted "X"	Cool, Sunny, 50° F
West drainage ditch	R.J. Hintz N
	N.R. Olson ϕ
	R.C. Perry ⊼
Summit	Wild Level #3
Sag	
Summit	COPY
	Page Check:
E gutter, Maple St.	+20.05
℄ Maple St.	−17.49
W gutter, Maple St.	+ 2.56
	360.48
	363.04
Summit	363.04 − 363.01 = Misclosure = 0.03
Top of E curb, Elm St.	
Bottom of E curb, Elm St.	
℄ Elm St.	
BM Store. NE corner Elm St. & 4th Ave. SE corner Store foundation wall. 3″ brass disc set in grout. BM store elev. = 363.01	R.J. Hintz

3. GRID, CROSS-SECTION, OR BORROW-PIT LEVELING

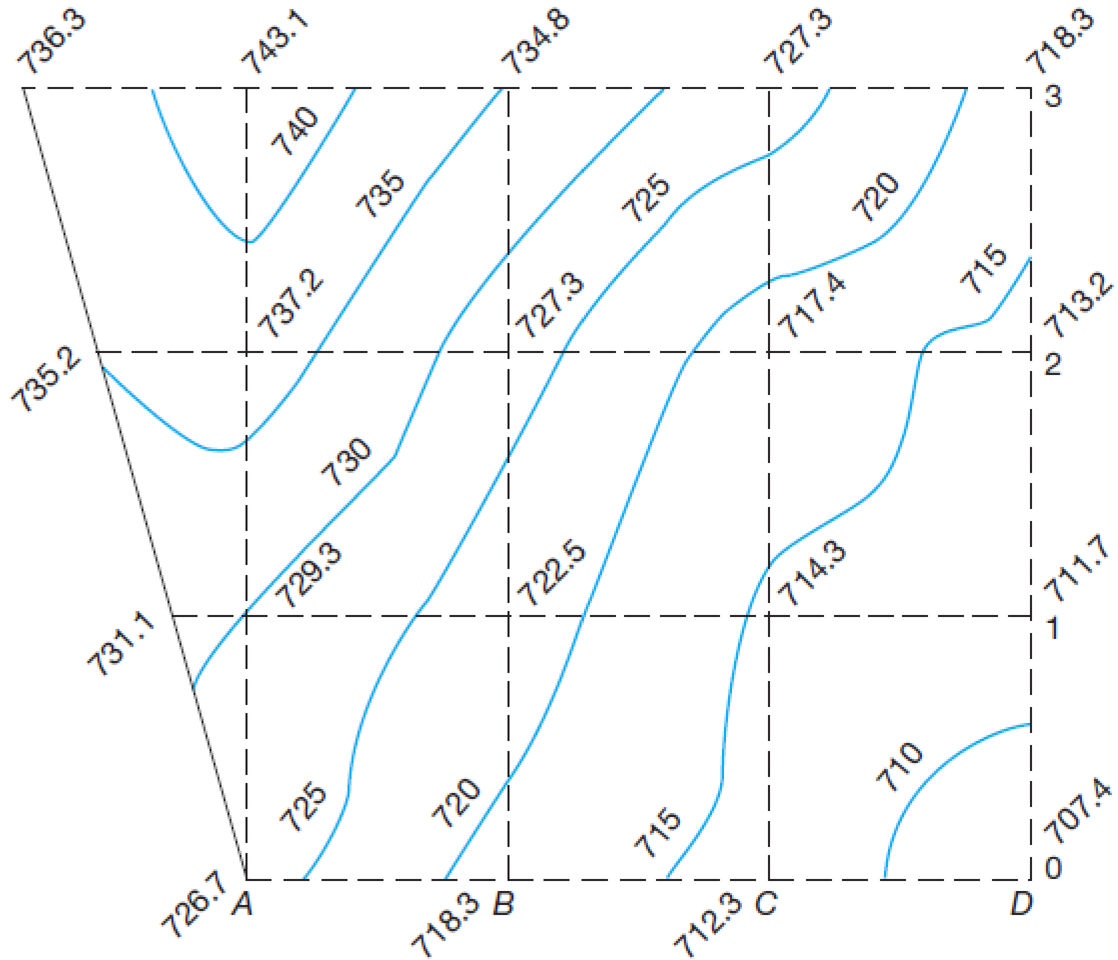
Grid leveling is a method for locating contours.

It is accomplished by staking an area in squares of 10, 20, 50, 100, or more feet (5, 10, 20, or 40 m) and determining the corner elevations by differential leveling.

The grid size chosen depends on the project extent, ground roughness, and accuracy required.

In plotting contours by the grid method, a widely spaced grid can be used for gently sloping areas, but it must be made denser for areas where the relief is rolling or rugged.

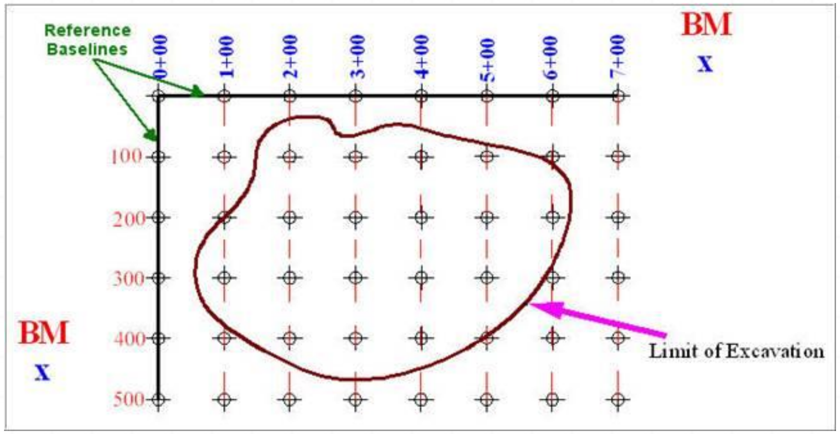
Contours are interpolated between the corner elevations (along the sides of the blocks) by estimation or by calculated proportional distances.



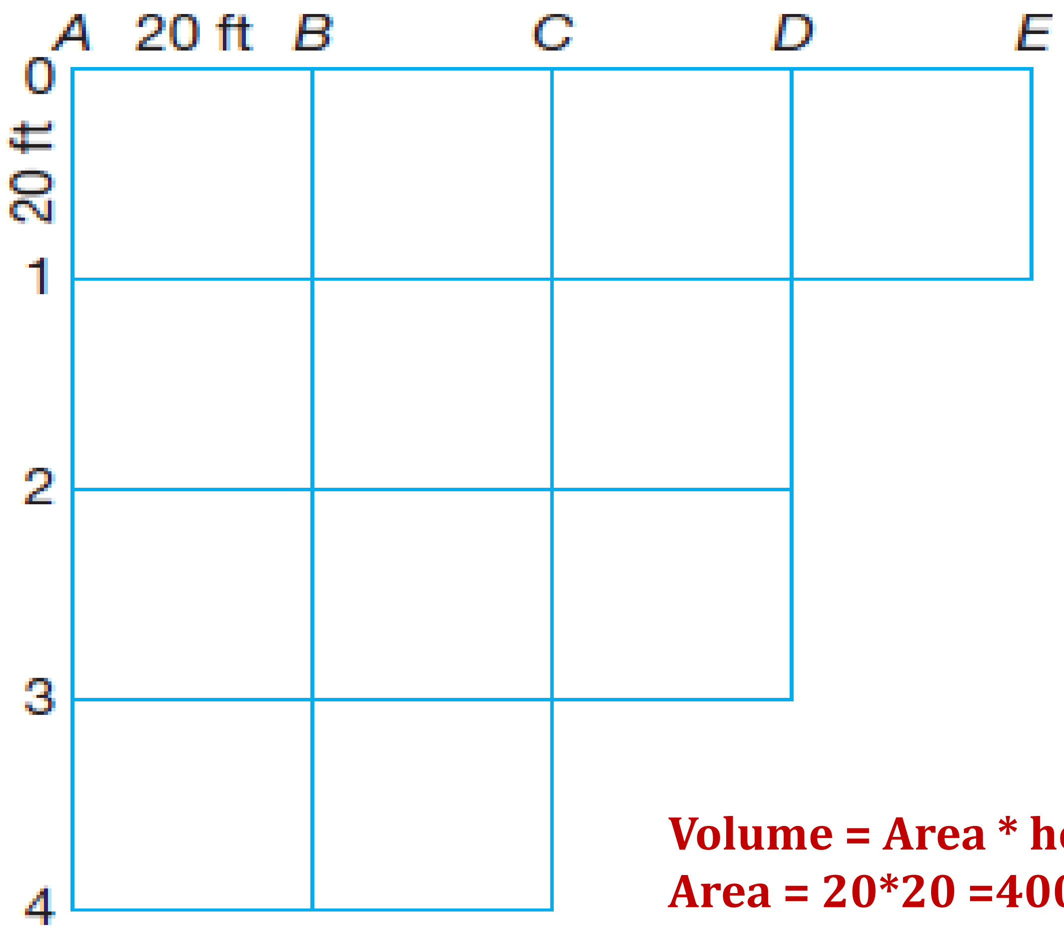
Grid Leveling

borrow-pit leveling Earth Volume Calculation

The level machine can be used to calculate volume of granular material (rock, gravel, sand), and volume of cuts and fills.



1- Establish one or two external reference baselines
2. Locate two external Bench Marks at convenient locations
3. Cross sections are established over and beyond the area
4. Establish datum X, Y, and Z coordinate points and get original volume of material.
5. Measure changes in Z coordinate and convert them to volume changes.



Borrow-pit leveling.

Volume = Area * height…
Area = 20*20 =400 ft----
height = Avg. of four readings per square area (sum hn/4)…
27 to convert cubic ft to cu yard.

Example:

BORROW-PIT LEVELING

Point	+ Sight	HI	− Sight	Elev.	Cut
BM Road	4.22	364.70		360.48	
A,0			5.2	359.5	1.5
B,0			5.4	359.3	1.3
C,0			5.7	359.0	1.0
D,0			5.9	358.8	0.8
E,0			6.2	358.5	0.5
A,1			4.7	360.0	2.0
B,1			4.8	359.9	1.9
C,1			5.2	359.5	1.5
D,1			5.5	359.2	1.2
E,1			5.8	358.9	0.9
A,2			4.2	360.5	2.5
B,2			4.7	360.0	2.0
C,2			4.8	359.9	1.9
D,2			5.0	359.7	1.7
A,3			3.8	360.9	2.9
B,3			4.0	360.7	2.7
C,3			4.6	360.1	2.1
D,3			4.6	360.1	2.1
A,4			3.4	361.3	3.3
B,4			3.7	361.0	3.0
C,4			4.2	360.5	2.5
BM Road			4.22	360.48	

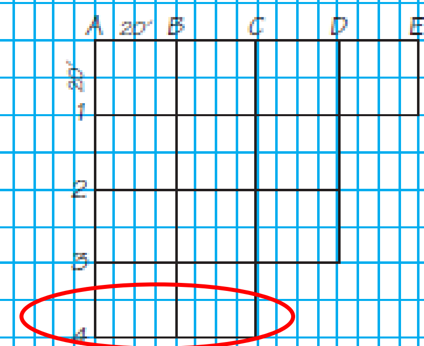
SECOND & OAK STREETS

hn

BM Road-Description p.5

Madison, WI
Cool, Cloudy, 60° F
B.A. Dewitt N
B.K. Harris ϕ
E.A. Custer ⊼
11 Oct. 2000
Kern Level #13

1.5
2.6
2.0
1.6
0.5
4.0
7.6
6.0
3.6
0.9
5.0
8.0
7.6
3.4
5.8
10.8
6.3
2.1
3.3
6.0
2.5



Grade elevation 358.0'

Volume = area of base x Σhn ÷ (4×27)

91.1 ÷ 4

$22.8 \times \frac{400}{27} = 337$ cu.yd.

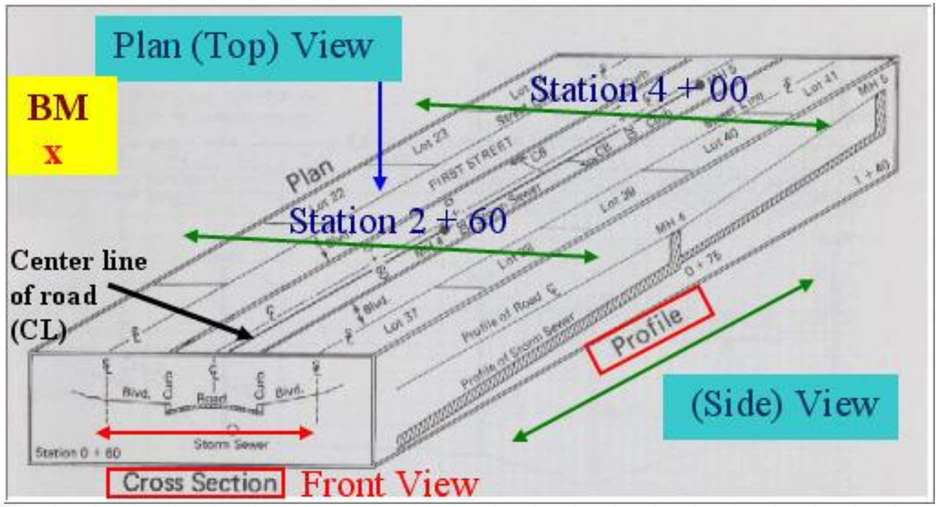
B.A. Dewitt

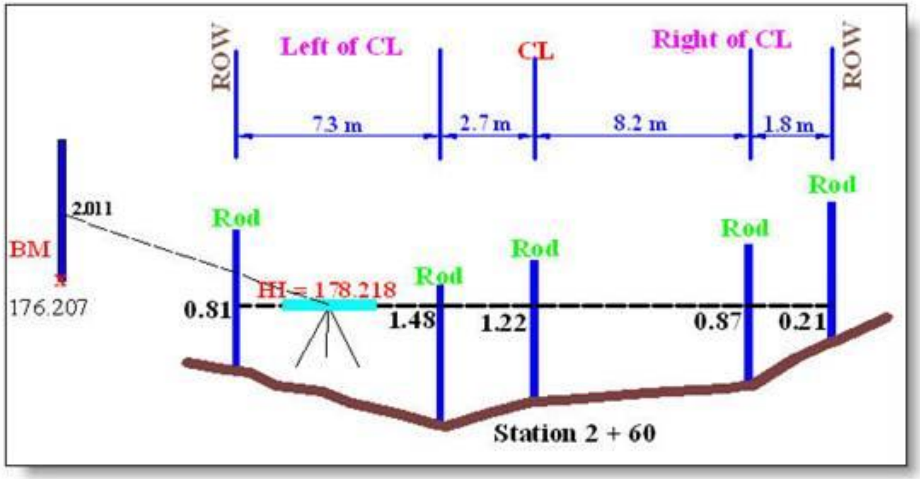
Plate B.2

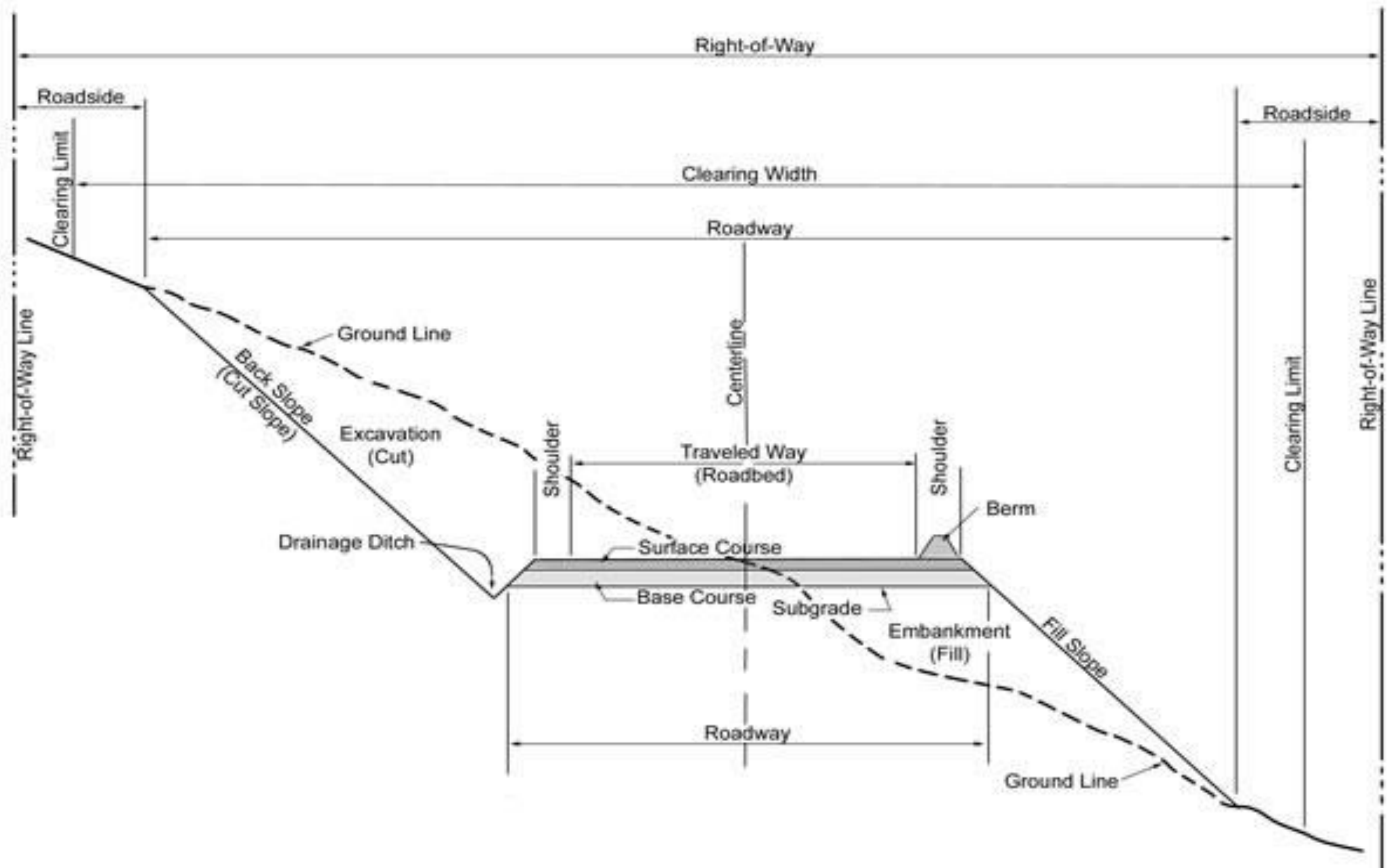
Cross-Section Leveling

Profile: is a side view or elevation of a certain area (surface of road or top of pipeline) in which the longitudinal surfaces are obtained. Profile levels are taken along a path (center line of a road) that is of interest to the engineer.

Cross section: shows the end view of a section at a certain station, and is at right angles to the center line.







Typical Road Section

Cross-Section Leveling

Station	BS (m)	IS (m)	FS (m)	HI (m)	Elevation (m)	Remarks
BM	2.011			178.218	176.207	Away from cross-sections
10 m left of CL		0.81			177.41	Station 2+60
2.7 m left of CL		1.48			176.74	Station 2+60
CL		1.22			177.00	Station 2+60
8.2m right of CL		0.87			177.35	Station 2+60
10 m right of CL		0.21			178.01	Station 2+60
10 m left of CL		1.02			177.20	Station 4+00
2.7 m left of CL		1.64			176.58	Station 4+00
CL		1.51			176.71	Station 4+00
8.2m right of CL		1.10			177.12	Station 4+00
10 m right of CL		0.43			177.79	Station 4+00

4. THREE-WIRE LEVELING

Three-wire leveling consists of making rod readings on the upper, middle, and lower crosshairs.

The method has the advantages of:

(1) Providing checks against rod reading blunders,
(2) Producing greater accuracy because averages of three readings are available, and
(3) Furnishing stadia measurements of sight lengths to assist in balancing backsight and foresight distances.

In the three-wire procedure the difference between the upper and middle readings is compared with that between the middle and lower values.

An average of the three readings is used as a computational check against the middle wire.

THREE-WIRE LEVELING
TAYLOR LAKE ROAD

Sta.	+ Sight	Stadia	− Sight	Stadia	Elev.
BM A					103.8432
	0.718		1.151		
	0.633	8.5	1.051	8.0	+0.6337
	0.550	8.3	0.972	7.9	104.4769
3	)1.901	16.8	3)3.154	15.9	−1.0513
	+0.6337		−1.0513		
TP1					103.4256
	1.151		1.041		
	1.082	6.9	0.969	7.2	+1.0820
	1.013	6.9	0.897	7.2	104.5076
3	)3.246	13.8	3)2.907	14.4	−0.9690
	+1.0820		−0.9690		
TP2					103.5386
	1.908		1.264		
	1.841	6.7	1.194	7.0	+1.8410
	1.774	6.7	1.123	7.1	105.3796
3	)5.523	13.4	3)3.581	14.1	−1.1937
	+1.8410		−1.1937		
BM B					104.1859
Σ	+3.5567	Σ	−3.2140		
Page	Check:				
	103.8432	+3.5567	−3.2140	=	104.1859 (Check)

Sample field notes for three-wire leveling.

5. Rise & Fall Method

- If the reading on
- A = 3.222 m
- B = 1.414 m,
- The difference in level between A & B = 3.222 – 1.414 = 1.808 m
- This represent a rise in the height of land at B relative to A.

- If the reading at B is greater than A, or of the direction of survey is from B to A, then the difference in elevation would represent a fall in the height of land at A relative to B.

- If the actual level of one of the two points is known, the level of the other may be found by either adding the rise or subtracting the fall.

Rise & Fall Example

Point #	B.S	I.S	F.S	Rise	Fall	Elevation	Remarks
BM 1	3.25					1050.17	BM 1050.17 m
A		3		0.25		1050.42	
B		2.85		0.15		1050.57	
C	1.82		2.75	0.1		1050.67	T.P.
D		2.13			0.31	1050.36	
E	3.16		0.78	1.35		1051.71	T.P
F		2.18		0.98		1052.69	
G		1.01		1.17		1053.86	
BM 2			0.68	0.33		1054.19	BM 1054.19 m
Sum	8.23		4.21	4.33	0.31		

Checking Computation in Rise & Fall

- The following conditions should be met:

The number of B.S. = The number of F.S.

Sum (B.S) – Sum (F.S) = Sum (Rises) – Sum (Falls)

= Last R.L – First R.L

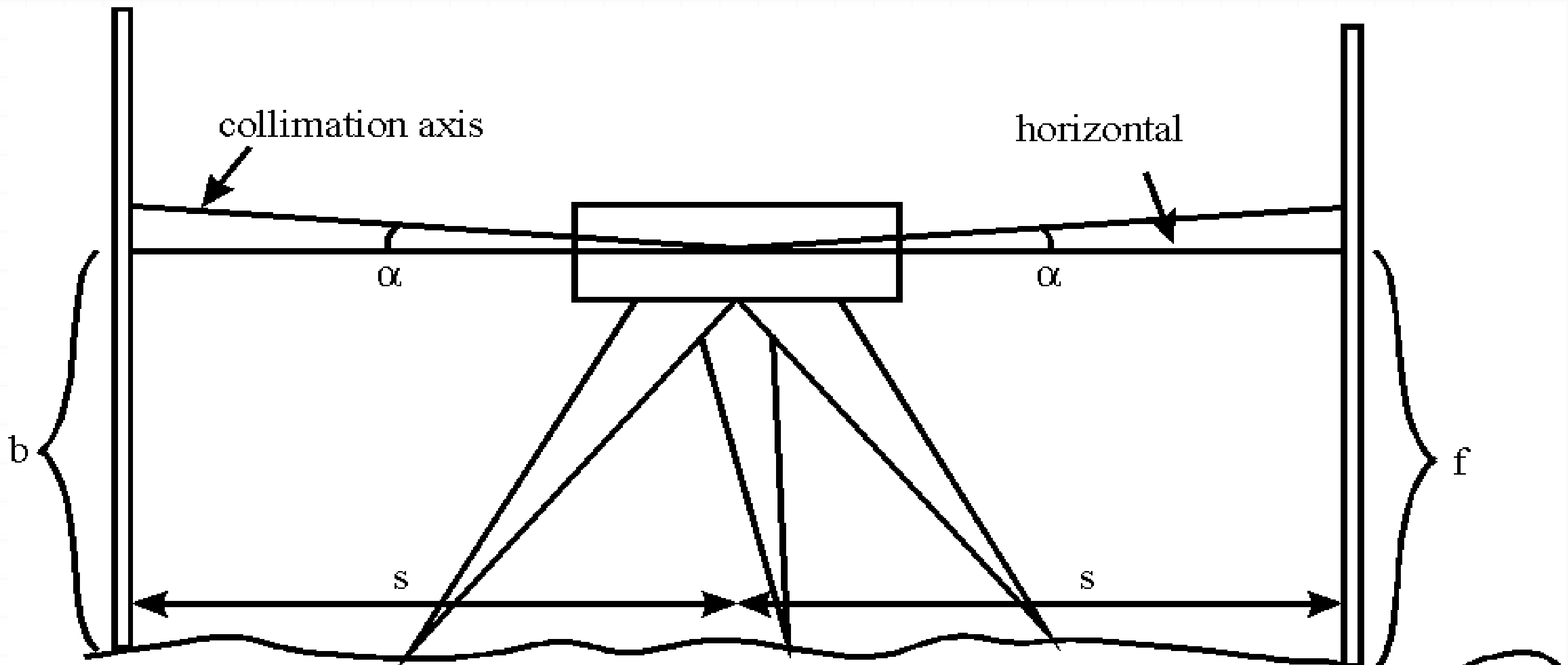
Checking Computation in Rise & Fall Example

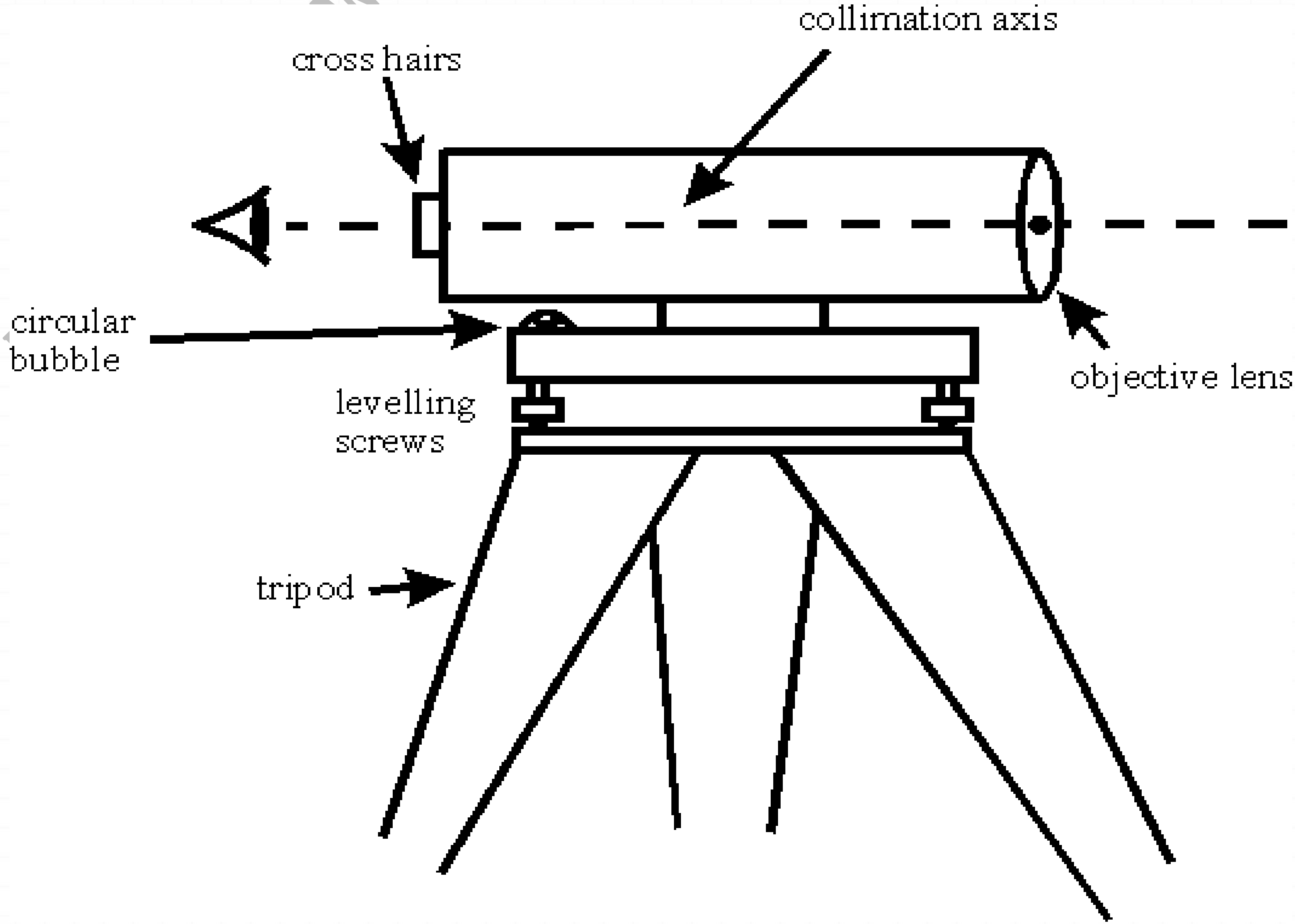
- No. of B.S readings = 3
- No. of F.S readings = 3
- 1st condition is O.K

- Sum (B.S) – Sum (F.S) =8.23 – 4.21 = 4.02 m
- Sum (Rises) – Sum (Falls) = 4.33 – 0.31 = 4.02 m
- Last R.L – First R.L = 1054.19 – 1050.17 = 4.02 m
- 2nd Condition is O.K

6. Collimation (Peg) Test

Collimation error occurs when the collimation axis is not truly horizontal when the instrument is level.
where the collimation axis is tilted with respect to the horizontal by an angle α.




collimation axis
cross hairs
circular
bubble
objective lens
levelling
screws
tripod

In this particular example, the effect is to read too high on the staff.

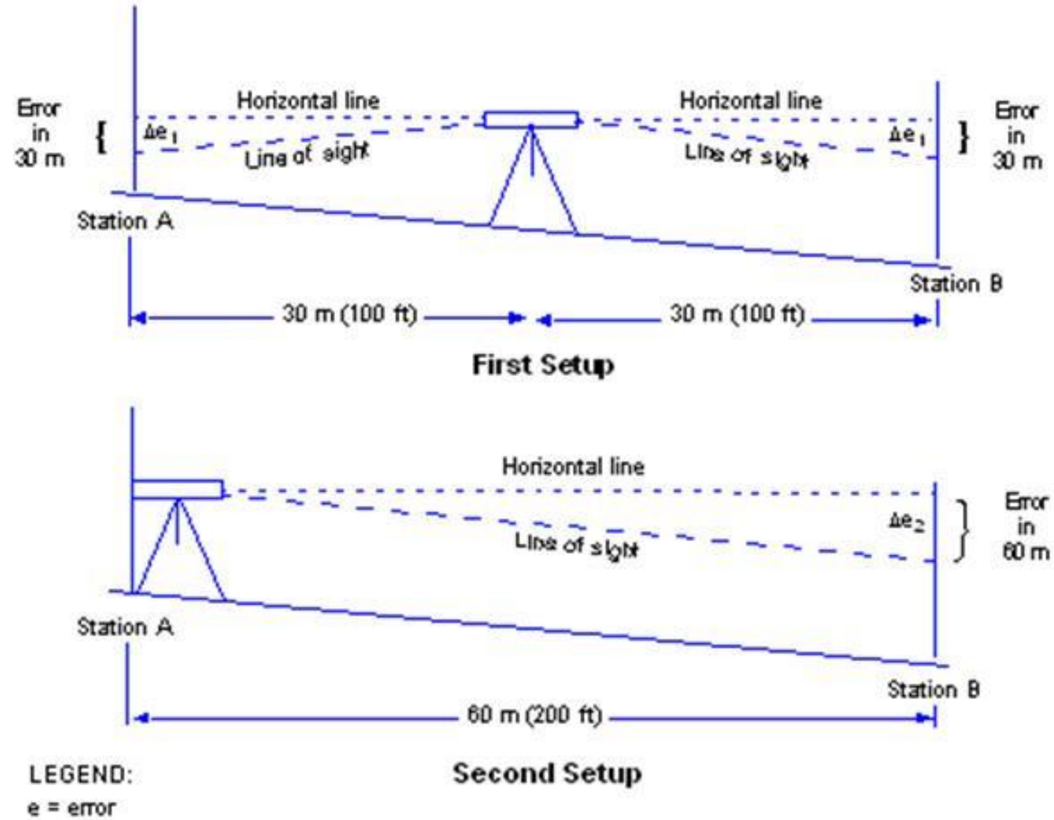
For a typical collimation error of 20", over a sight length of 50m the effect is 5mm.

If the sight lengths for backsight and foresight are *equal*, the linear effect is the same for both readings.

When the height difference is calculated, this effect cancels:

$$\delta h = (b + s.\alpha) - (f + s.\alpha)$$
$$= b - f$$

That is, the effect of the collimation error is eliminated if sight lengths are kept equal.



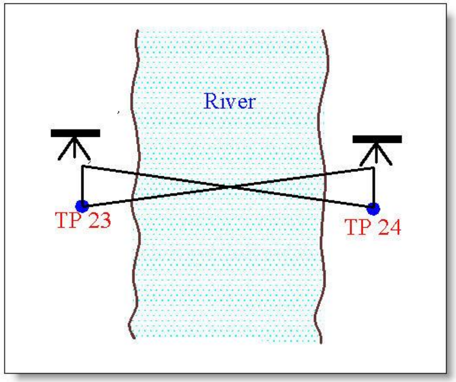
DETERMINATION OF COLLIMATION ERROR

Collimation error is much more significant than the other errors. It should be kept as small as possible so that one need not be too precise in ensuring that fore- and back sights are of equal length (these are usually paced out). It is possible to determine the collimation error and reduce its size using the so-called **Two-peg test.**

1. Place the level midway between two rods which are 60 m apart.
2. Take Rod Readings for both rods. If line of sight is not horizontal, the error in rod readings (Δe_1) will be identical.
3. Calculate difference in elevation between A & B, this is the correct difference since both stations have equal Δe_1.
4. Move level close to Station A. Get difference in elevation between A & B.
5. Calculate the difference in the readings (Δe_2) from point 3 (true difference in elevation) and point 4 (difference in elevation containing error).
6. Divide De_2 by the total distance (60m). Obtained value in (m/m) is called collimation factor (C factor).

- ***To eliminate the effect of the collimation error, the leveling machine is placed midway between the BS and FS.***

7. Reciprocal leveling



In the previous section it was mentioned that *"To eliminate the effect of the collimation error, the leveling machine is placed midway between the BS and FS"*.

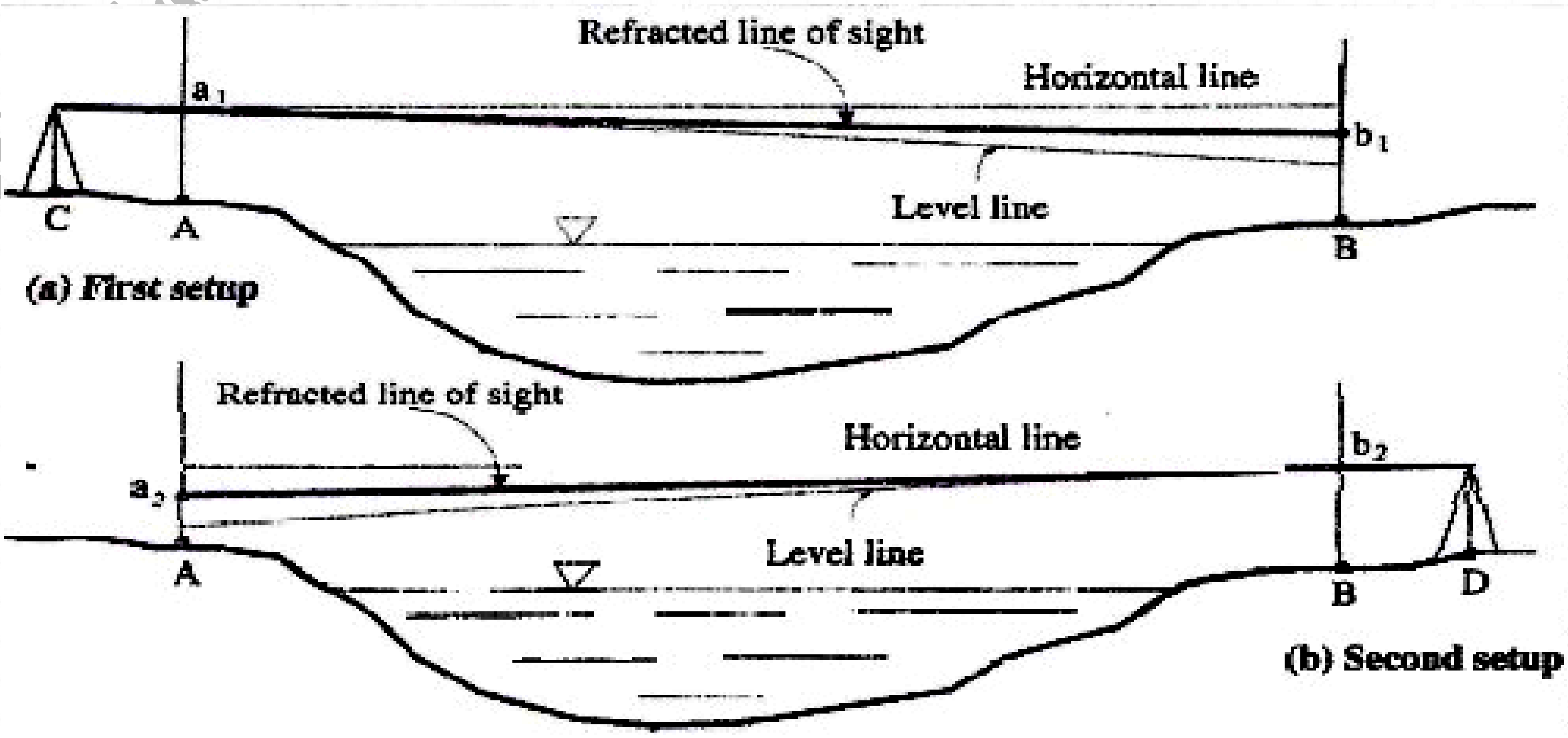
Sometimes it is difficult to place the level between the BS & FS stations due to presence of an obstacle (street, valley or river). In this case we have to use the reciprocal leveling.

1-The level is setup in position 1 and Rod Readings are taken at TP23 and TP24. Then the difference in elevation between the tow points is calculated.

2. The level is moved to position 2 and Rod Readings are taken at TP23 and TP24. Then the difference in elevation between the tow points is calculated.

3. The obtained differences in elevation at the two positions are averaged to get the true difference in elevation.

Reciprocal leveling Example



1) Set up the level at point C (Figure 4.15a), 2 to 3 m from A and take the readings a_1 at A and b_1 at B. Calculate the first elevation difference:

$$\Delta H_1 = a_1 - b_1$$

2) Move the level to point D (Figure 4.15b) so that the distance AC = BD. Take the two readings a_2 at A and b_2 at B. Calculate the second elevation difference:

$$\Delta H_2 = a_2 - b_2$$

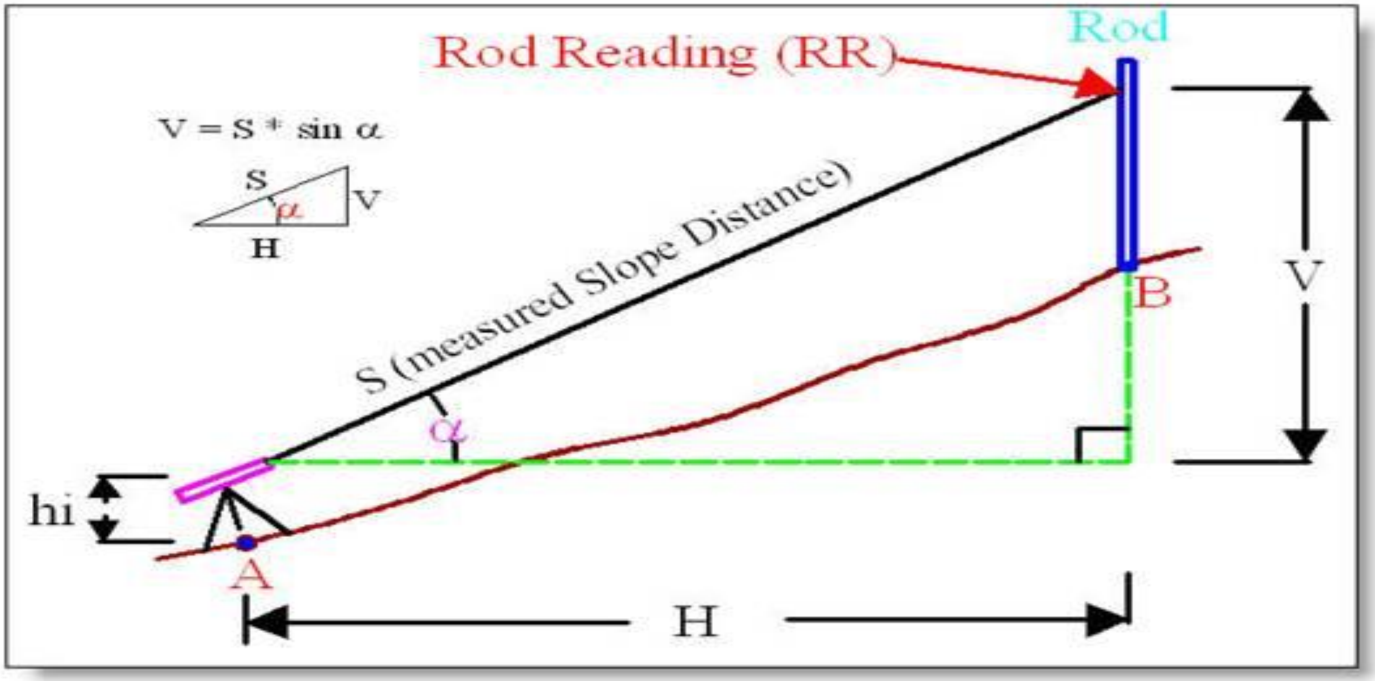
Calculate the correct elevation difference (ΔH) as follows:

$$\Delta H = \frac{\Delta H_1 + \Delta H_2}{2} = \frac{(a_1 - b_1) + (a_2 - b_2)}{2} \quad \ldots\ldots$$

8. Trigonometric leveling

The Theodolite (angles measuring equipment) can also be used to calculate differences in elevations. There are three cases for the target point position:

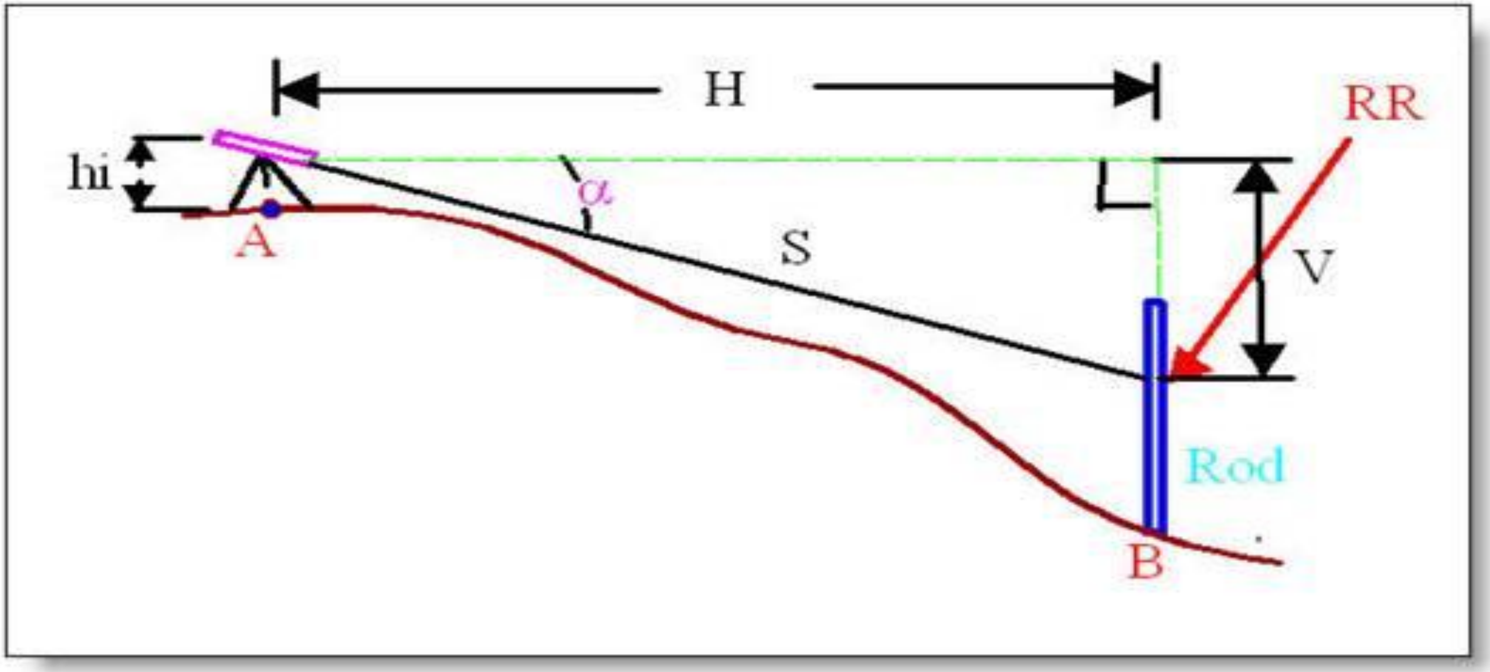
1. Target point is higher than the Theodolite



V = S sin α

Elevation of B (Rod position) = Elevation of A (Theodolite position) + hi + V - RR

2. Target point is lower than the Theodolite:



V = S sin a

Elevation of B (Rod position)= Elevation of A (Theodolite position) + hi - V - RR

3. Target point is at the same level of the Theodolite ($\alpha = 0.000$)

In this case the Theodolite is handled similar to the level because now it is horizontal.

$V = S \sin \alpha = 0$

Elevation of B (Rod position) = Elevation of A (Theodolite position) + hi - RR

Prof. TALEB AL-ROUSAN

Surveying

2104011365

Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University

Prof. TALEB AL-ROUSAN

Chapter 4
Theodolites

Angles measured in surveying are classified as either *horizontal* or *vertical*, depending on the plane in which they are observed.

Horizontal angles are the basic observations needed for determining bearings and azimuths.

Vertical angles are used in trigonometric leveling, stadia, and for reducing slope distances to horizontal

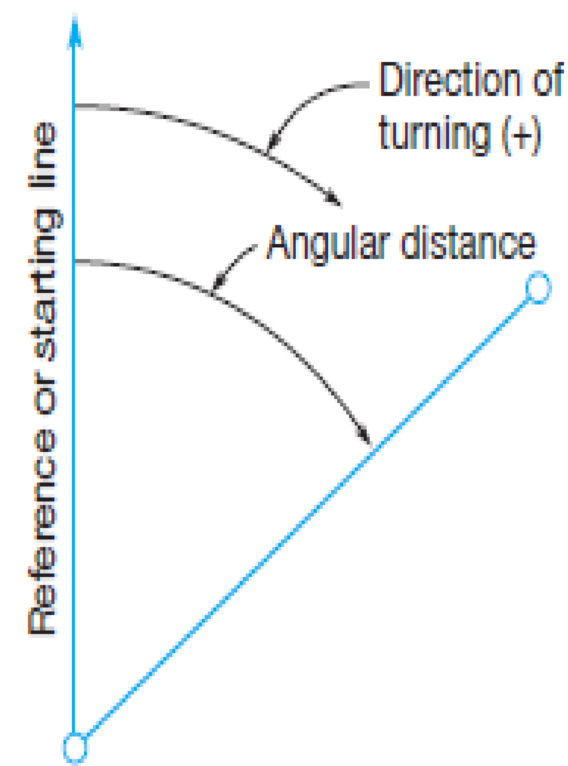
Angles are most often directly observed in the field with total station instruments, although in the past transits, theodolites, and compasses have been used.

Three basic requirements to determine an angle are:

(1) reference or starting line,

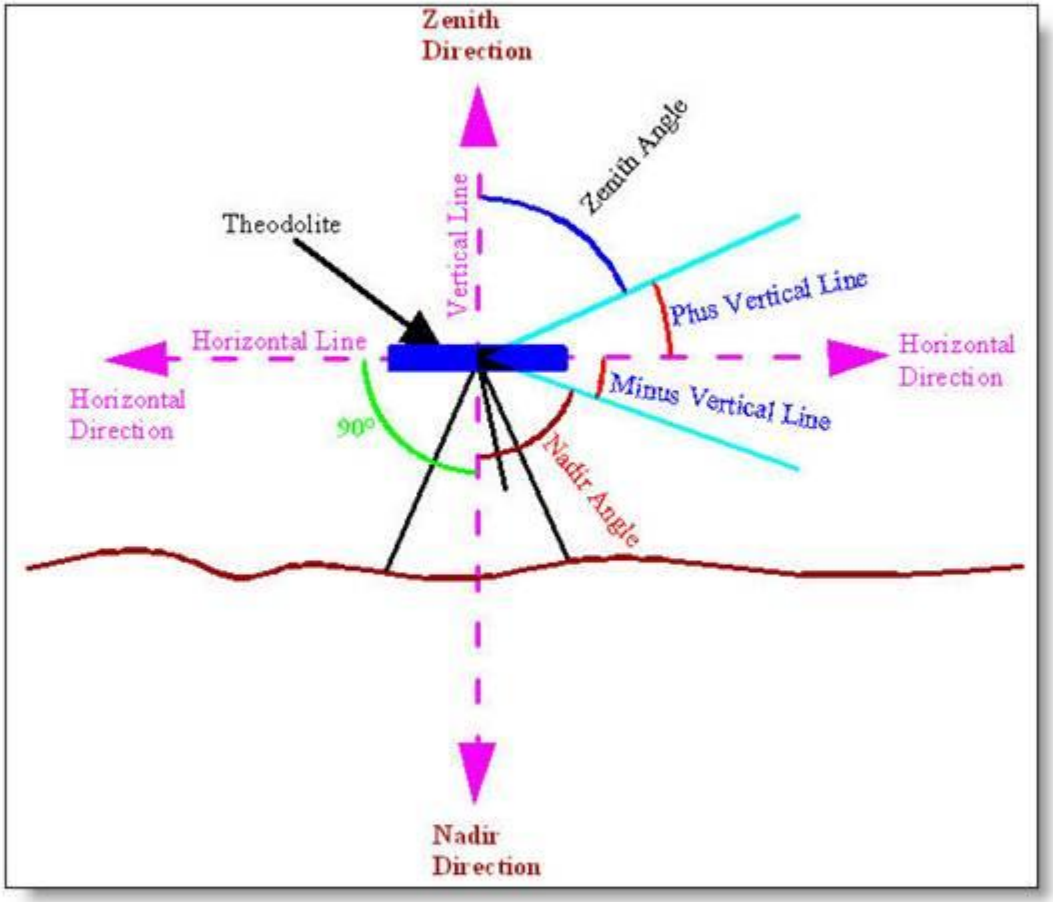
(2) direction of turning, and

(3) angular distance (value of the angle).



Vertical Angles are referenced to:

- The horizon by plus (up) or minus (down) angles.
- The zenith: directly above the observer.
- The nadir: directly below the observer.



Theodolites/ Transits

Older versions were called Transits. Nowadays, both words (Transits and Theodolites) are used interchangeably.

Usage:

- **Measure horizontal angles (deviation from the North).**
- **Measure vertical angles (deviation from horizon, Nadir, or Zenith).**
- **Establish straight lines.**
- **Establish horizontal and vertical distances by using stadia.**
- **Establish difference in elevation when used as leveling machine.**

Theodolites:

Theodolites (sometimes called transits) are used in measuring horizontal and vertical angles and for establishing linear and curved alignments in the field.

⇒ **Targets: are plates having their center marked.**

Types of Theodolites

Types of Theodolites

⇒ **In terms of measuring operation:**

- **Repeating instruments:**
  - Can be zeroed, measure 1α, 2α, 3α, ...
  - The circle assembly has two clamps (upper & lower)

- **Direction instruments:**
  - Can not be zeroed.
  - The circle assembly has just one clamp (upper)

⇒ **In terms of model:**

- **Engineer transit:**

→ Old

→ USA

→ Horizontal setting 0° zenith

Types of Theodolites

- Optical theodolite (Repeating):
  - → New
  - → USA & other countries
  - → Horizontal setting 90° or 270° zenith
  - → 0° zenith could be at the zenith or Nadir

- Electronic Theodolites:
  - → Similar to optical theodolites
  - → Precision is high
  - → Digital readouts (no interpolation)
  - → Zero-set buttons
  - → Horizontal angles can be turned left or right
  - → Automatic repeat - angle averaging
  - → Add EDM ⇒ Total Station

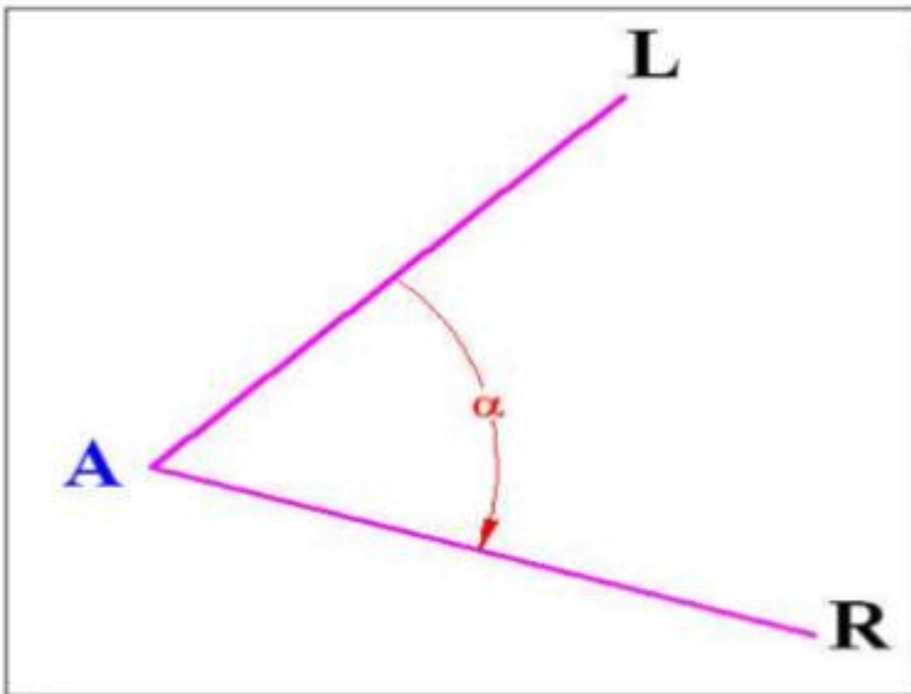
How to check if the theodolite is measuring in Nadir, Zenith, or from Horizon?

Put telescope in a horizontal position and tilt it slightly up and check reading:

- **If reading close to zero Þ Reading from horizon**
- **If < 90 Þ Zenith**
- **If > 90 Þ Nadir**

Measuring Horizontal Angles

Turning the angle at least twice (plunging\transiting the telescope) will eliminate mistakes, most instrument errors, and increase precision.



Directional Theodolites:

(Directional Theodolites can't be zeroed)

- **1. Theodolite at A**
- **2. While instrument at Face-Left (FL), vertical circle on the left side of surveyor, target telescope at "L" point and record reading in the column FL *(a)* corresponding to point L.**
- **3. Go clockwise and target at "R" point and record reading in the column FL *(b)* corresponding to point R. The difference in the readings in the "FL" Column will be nearly equal to the value of the angle.**
- **4. Plunge (transit) the telescope, now the instrument is Face Right (FR), vertical circle on the right side of surveyor.**
- **5. While still targeting on "R", record reading in the column FR corresponding to point R *(c)*. The difference between the FL and FR readings for the same point should be around 180°.**

Directional Theodolites:

(Directional Theodolites can't be zeroed)

- **Go anticlockwise and target on point "L" and record reading in the column FR corresponding to point L.**

- **8. In the "Mean" column, take the mean of the minutes and seconds for each point and take the degrees for that point either from the FL or FR column. You have to stick to one of the positions, FL or FR, in the whole table for taking the degrees values.**

- **9. The angle value is calculated by getting the difference between the two values in the "Mean" column.**

Measuring Horizontal Angles

ST	PT	Position I (FL)	Position II (FR)	Mean	Angle
A	L	276° 14'23" (a)	96° 14' 34" (d)	276° 14' 28"	31° 37' 09"
	R	307° 51' 33" (b)	127° 51' 41" (c)	307° 51' 37"	

Directional Theodolites:

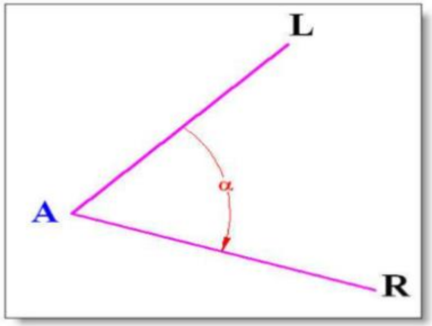
(Directional Theodolites can't be zeroed)

- **10. The angle LÂR can be obtained by calculating difference in angels from position I (FL) and position II (FR).**

ST.	PT.	POSITION I (F.L.)	POSITION II (F.R.)
A	L	276° 14' 22"	96° 14' 34"
	R	307° 51' 33"	127° 51' 41"
	Difference	31° 37' 11"	31° 37' 07"
	Mean		31° 37' 09"

Repeating Theodolites

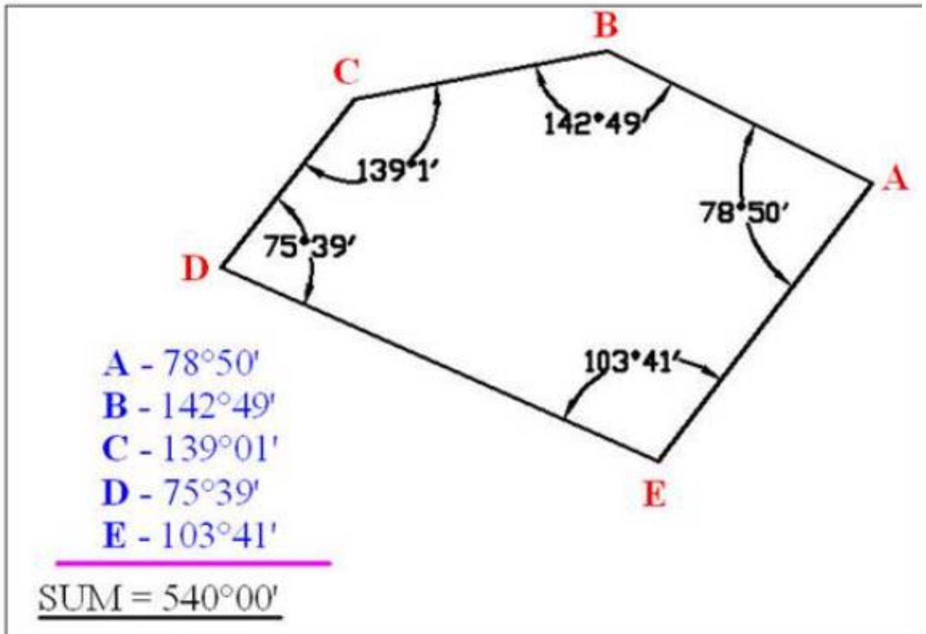
(Repeating theodolites can be zeroed)



- **1. Theodolite at A**
- **2. Zero instrument and target L**
- **3. Go clockwise and target R**
- **4. Record in (Direct)**
- **5. Plunge telescope disengage lower motion gear**
- **6. Target at L, record in (Direct)**
- **7. Go clockwise and target R and record double**
- **8. Take mean of double**

ST	Direct	Double	Mean = Angle
A	13° 20'12"	26° 40' 28"	13° 20' 14"

Example



ST	Direct	Double	Mean = Angle
A	78° 49’23”	157° 39’ 08”	78° 49’ 34”
B	142° 49’53”	285° 38’ 28”	142° 49’ 14”
C	139° 00’17”	278° 01’ 56”	139° 00’ 58”
D	75° 39’12”	151° 17’ 56”	75° 38’ 58”
E	103° 41’10”	207° 22’ 28”	103° 41’ 14”
		Summation	539° 59’ 58”

Correct Summation of angles = (n-2) * 180 = 3 * 180 = 540° 00’

Angular error of closure = 540° 00’ 00" - 539° 59’ 58” = 02"

Measuring Vertical Angles

Vertical angles are angles measured in the vertical plane with zero or reference being a horizontal or a vertical line. That is, a vertical angle is not measured from a low point to a high point, but from the horizontal to the high point, a ($+^{ve}$) vertical angle or an angle of elevation, and from the horizontal to the low point, a ($-^{ve}$) vertical angle or an angle of depression.

Vertical angles are referred to the vertical line in modern instruments and called zenithal angles (or zenithal distances). If the angle lies between 0° and 90°, it is an angle of elevation ($+^{ve}$), otherwise it is an angle of depression ($-^{ve}$) (between 90° and 180°).

Vertical angles are subject to index error which results from:

a. Displacement of the vertical circle

b. Lack of adjustment of the vertical circle reading device.

The index error is eliminated by sighting in two positions.

Measuring Vertical Angles

Measurement Procedure

1. Sight while the theodolite is in position I (Face Left) with the horizontal hair bisecting the target.

2. Center the bubble of the index level (match both ends in case of split bubble levels). This step is not needed in theodolites with automatic vertical collimation.

3. Take the reading and record it (87° 22' 43").

4. Reverse the telescope to position II (Face Right) and repeat steps 1, 2, and 3. Record the reading (272° 39' 57").

5. Add both readings and compare the results with 360°. The difference (0° 2' 40") is twice the value of the index error.

Measuring Vertical Angles

Measurement Procedure

6. Correct the readings such that their sum agrees with 360° exactly (87° 21’ 23” + 272° 38’ 37”= 360° 00’ 00”).

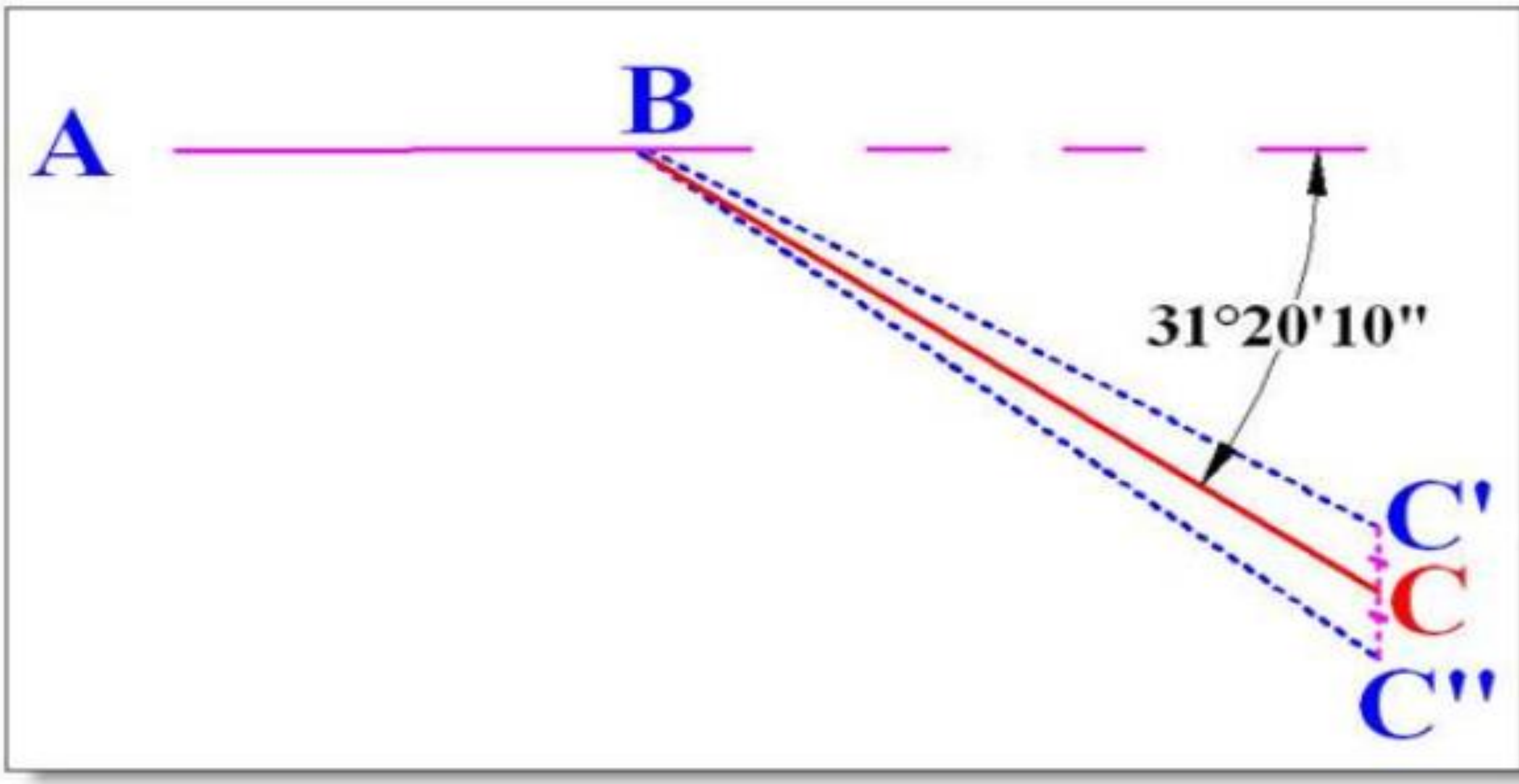
7. Subtract the corrected angle of position I from 90° to get the vertical angle (90° 00’ 0” - 87° 21’ 23” = + 2° 38’ 37”).

PT.	POSITION I	POSITION II	SUM	INDEX ERROR	VERTICAL ANGLE
P5	87° 22’ 43” 87° 21’ 23”	272° 39’ 57” 272° 38’ 37”	360 2’ 40” 360° 00’ 00”	- 0° 1’ 20”	+2° 38’ 37”

Field Applications

i) Laying out external angles:

Given Line AB ⇒ required to layout line BC @ 31°20'10"



Field Applications

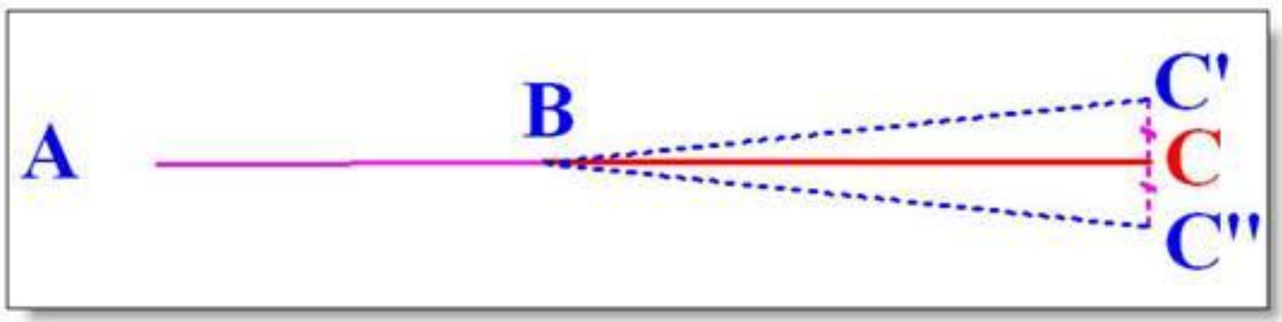
Procedure:

1- Set Theodolite at B.
2- Sight on A and zero Theodolite.
3- Plunge (transit) telescope and turn required angle (31°20'10").
4- Locate point C'.
5- Hold the angle reading and sight again on A.
6- Release the angle reading, plunge the telescope and turn it the required angle again. The reading will be double the angle value (62°40'20").
7- Locate point C".
8- Locate point C which is midway between C' and C".

Field Applications

Prolonging straight lines

Given Line AB ⇒ required to prolong the line



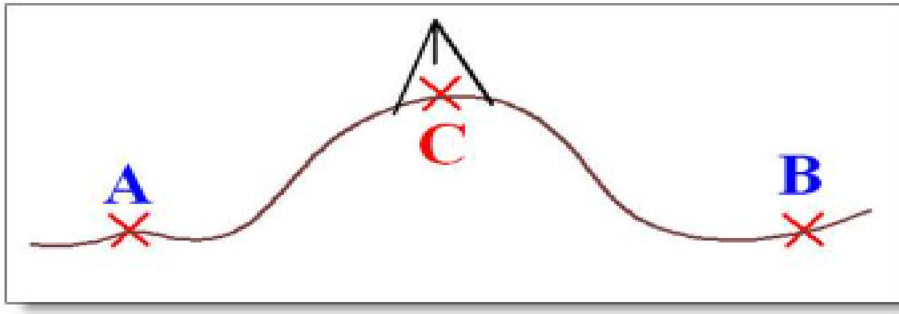
Procedure:

1- Set Theodolite at B and sight on A.
2- Plunge telescope and locate point C'.
3- Rotate telescope and sight again on A.
4- Plunge telescope and locate point C".
5- Locate point C which is midway between C' and C".

Field Applications

III. Interlining (Balancing in)

Interlining is establishing a straight line between two not inter-visible points.



Procedure:

1- Select a point "C" between the two points that you can see both points from it.

2- Roughly align C' between A & B.

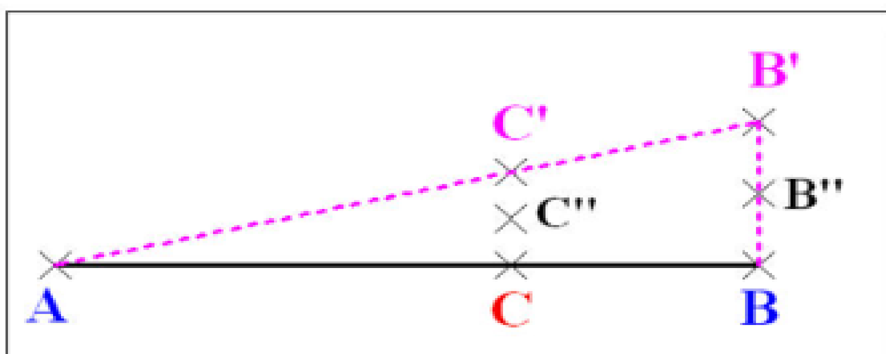
3- Sight at A.

4- Plunge telescope and locate point B'.

5- Measure BB' & calculate CC'.

6- Shift Theodolite to C''.

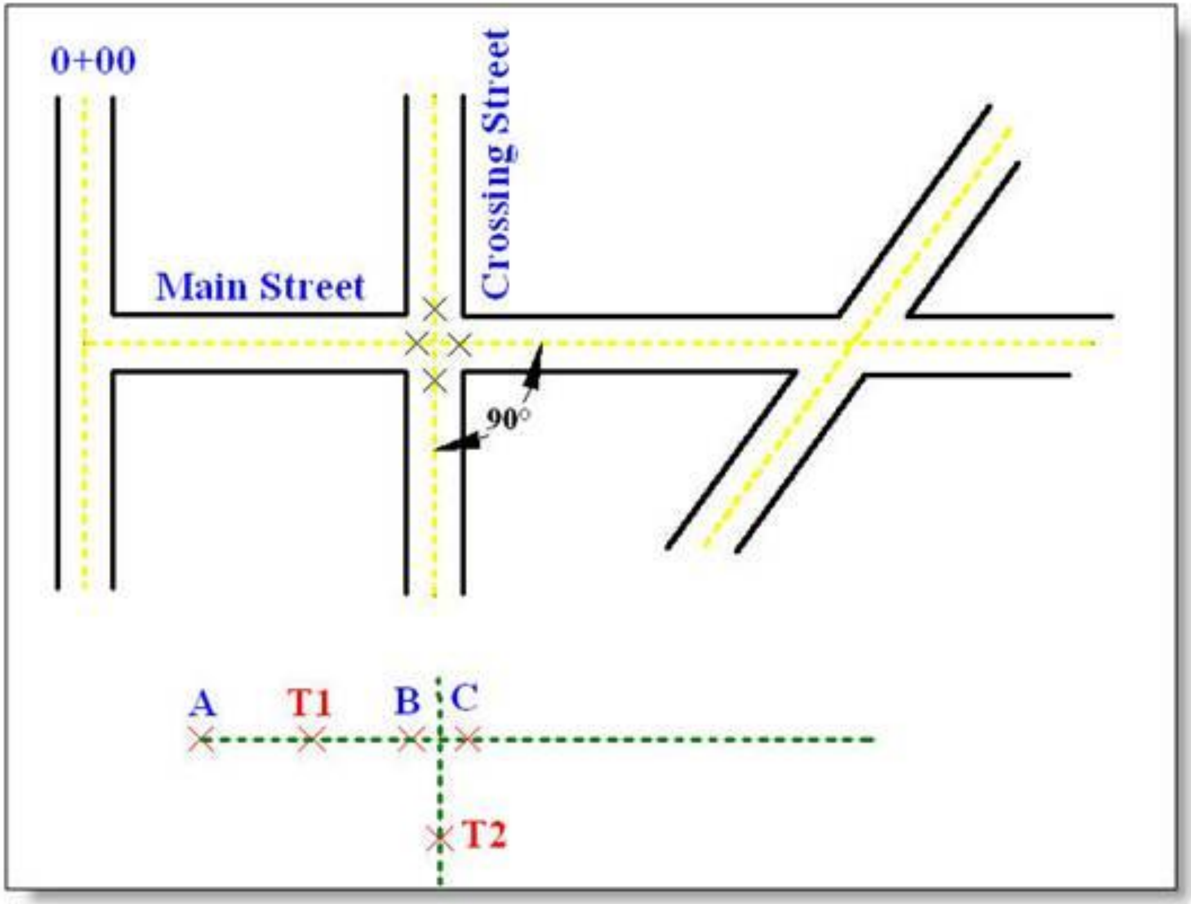
7- Repeat steps 3 – 6 until A, B & C are aligned.



Field Applications

- **Intersection of two straight lines**

- **This is a common situation when the surveyor is required to locate the intersection point when laying out intersecting streets with the main street.**



Field Applications/ Intersection of two straight lines

Procedure:

1- Set Theodolite @ main street center line (T1) and locate tow points (B & C) on it (about 1 m apart) to be on both sides of the intersection point.

2- Stretch a string between points B & C.

3- Move the Theodolite to the center line of the crossing street (T2).

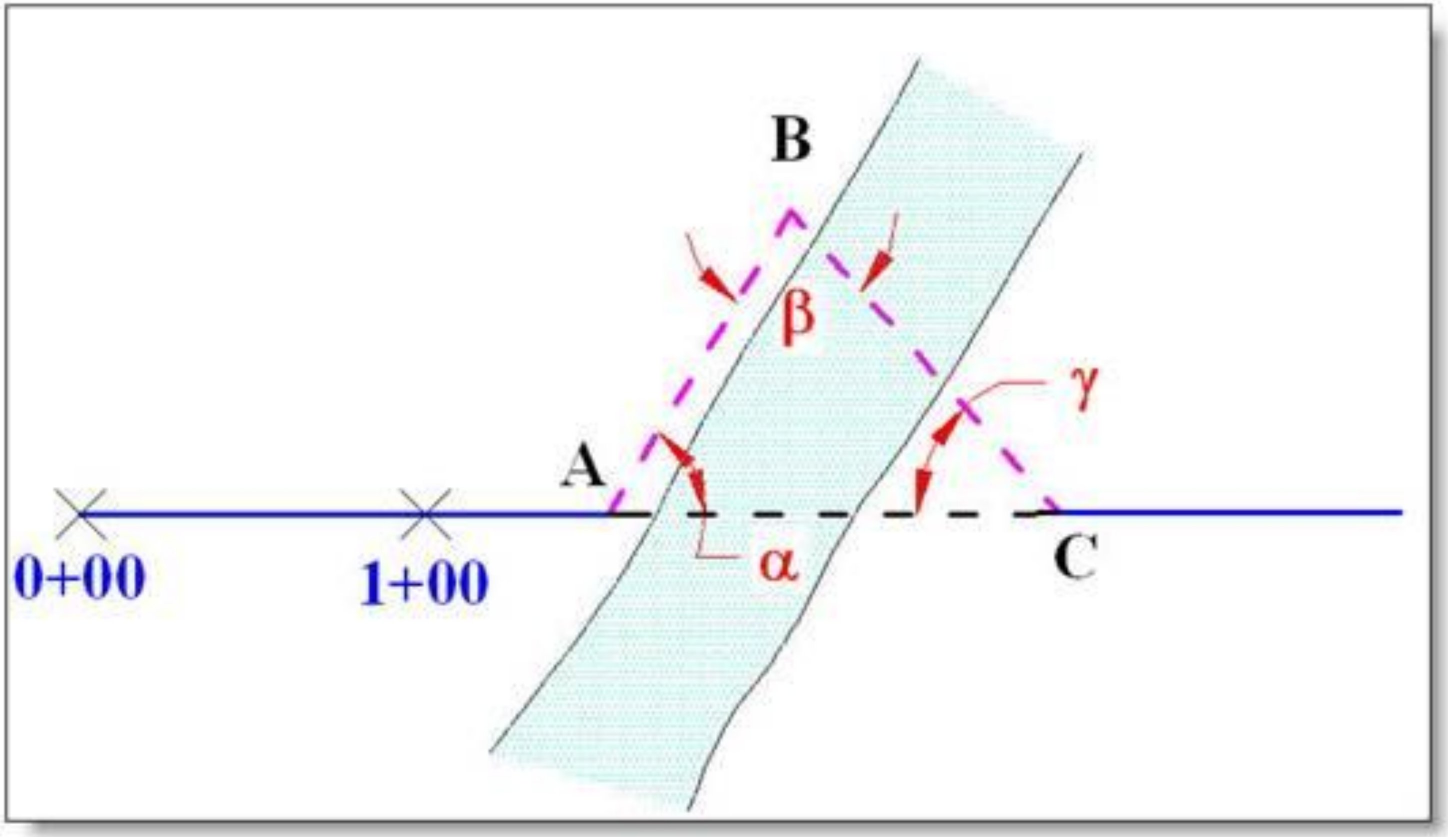
4- Align the telescope with the center line.

5- The instrument man is now in a position to find the intersection point between his view sight and the stretched string.

Field Applications

Chainage of a straight line over an obstacle

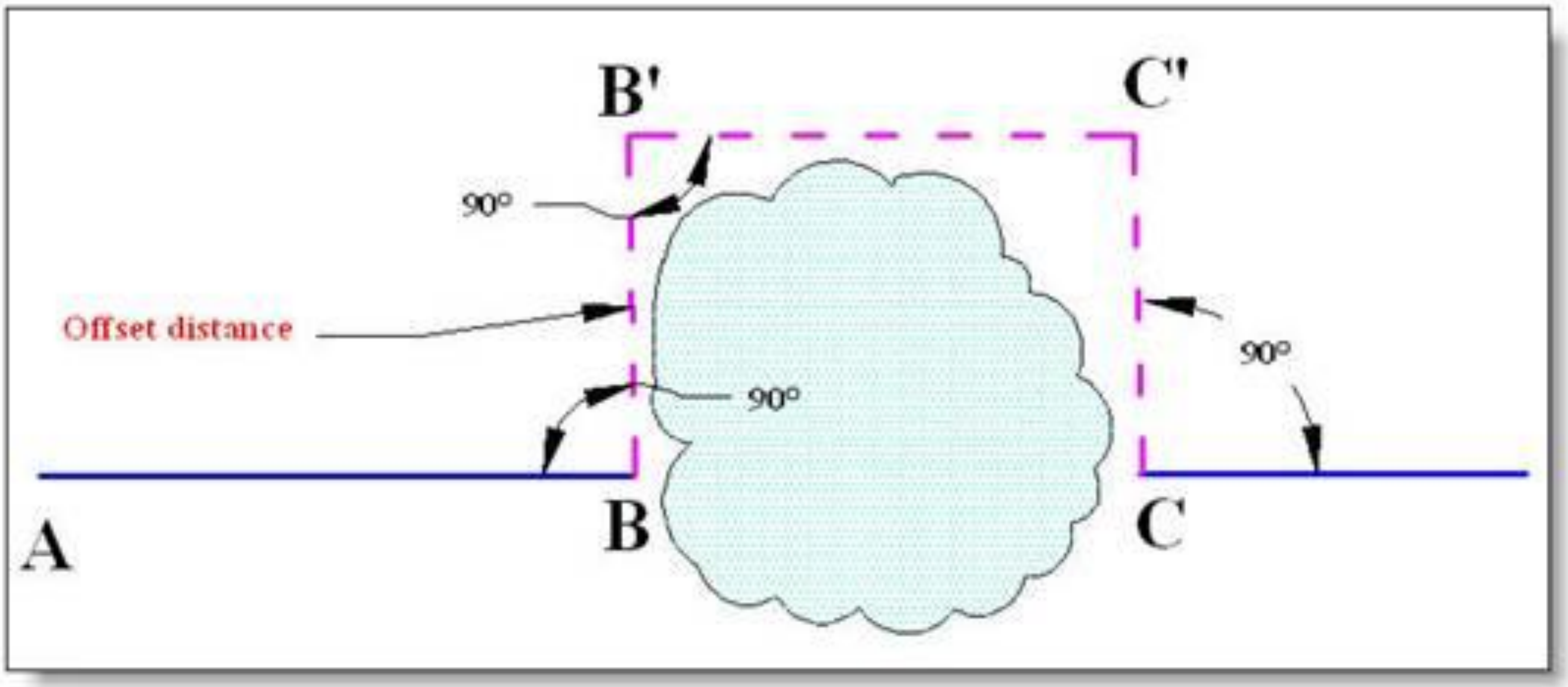
Chainage, locating stations, on a straight line when an obstacle is faced can be continued using triangulation (Sine law)



Field Applications

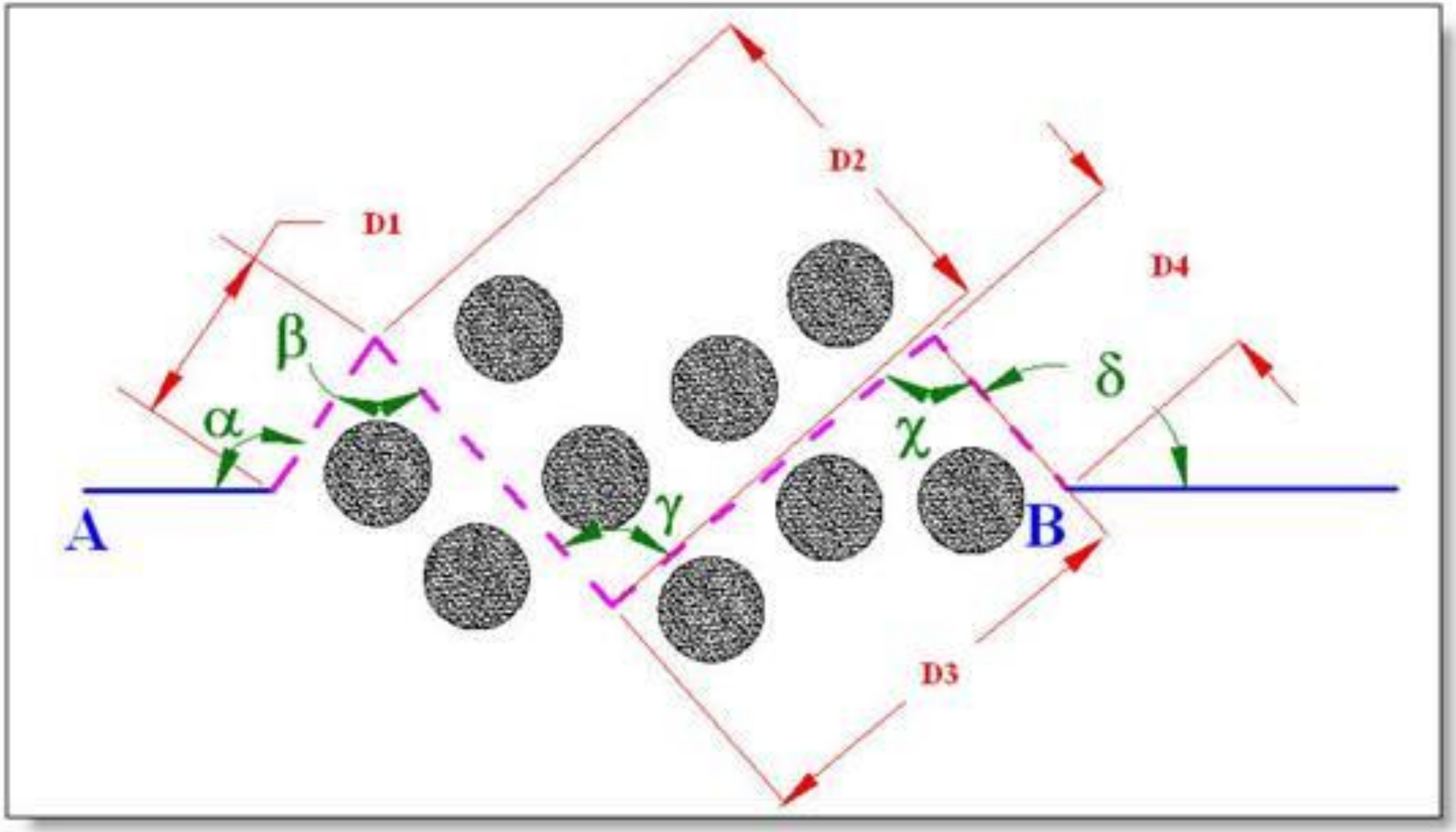
Prolonging a line past an obstacle

Right angle offset



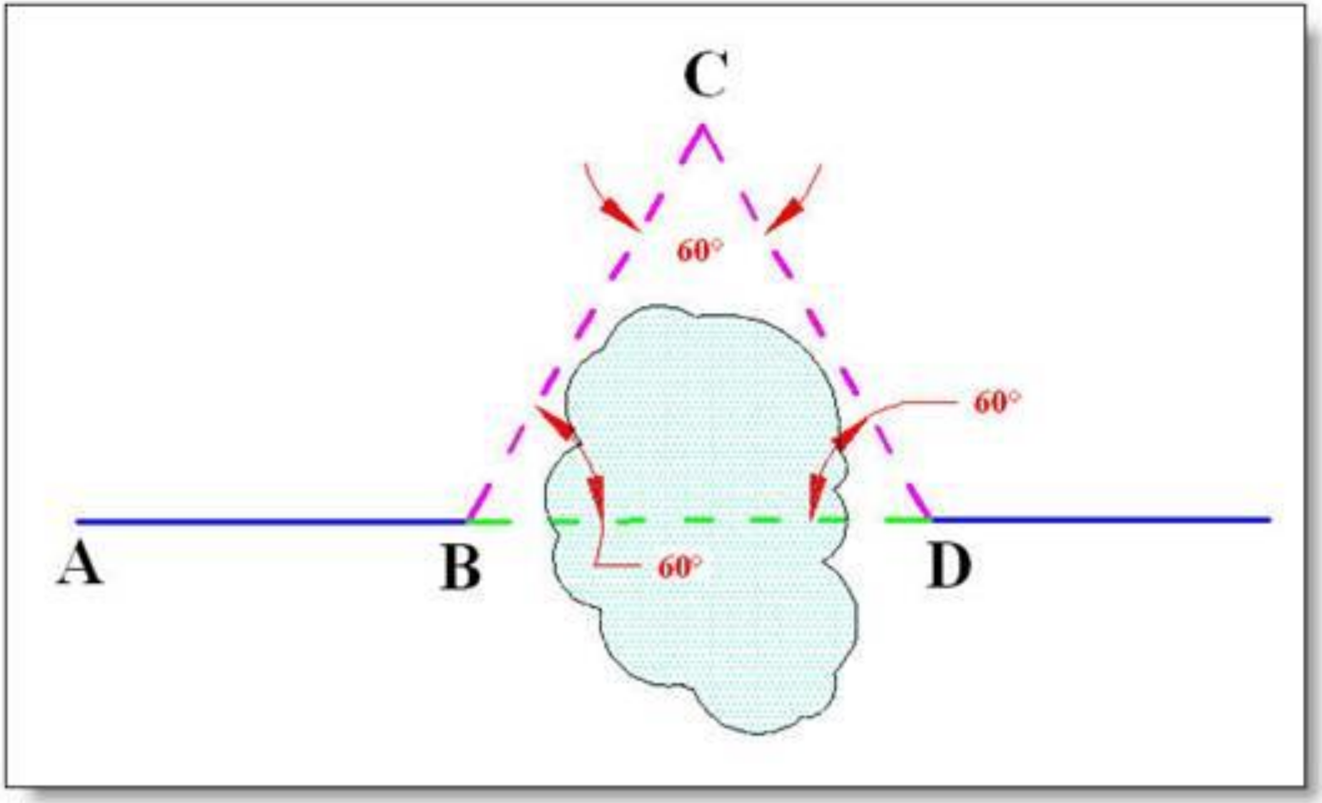
Field Applications

Random-line method



Field Applications

Triangulation Method/ Using equilateral triangle.



Prof. TALEB AL-ROUSAN

Surveying

2104011365

Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University

Prof. TALEB AL-ROUSAN

Chapter 5
Angles & Directions

For Reminder:

- Determining the locations of points and orientations of lines frequently depends on the observation of angles and directions.

- Angles measured in surveying are classified as either *horizontal* or *vertical*, depending on the plane in which they are observed.

- Horizontal angles are the basic observations needed for determining bearings and azimuths.

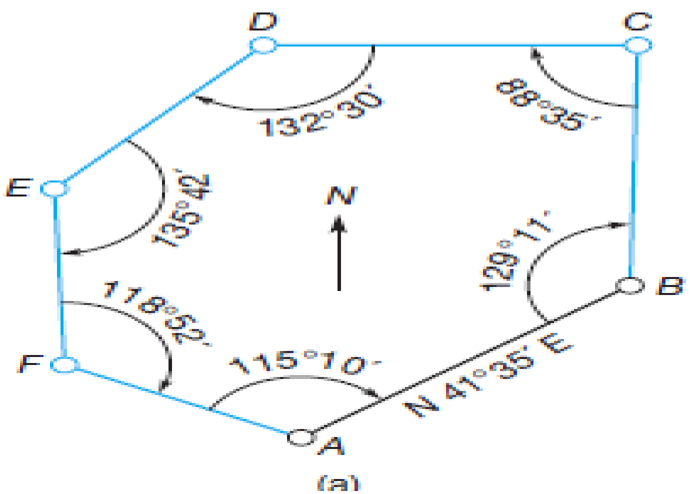
- Vertical angles are used in trigonometric leveling, stadia, and for reducing slope distances to horizontal

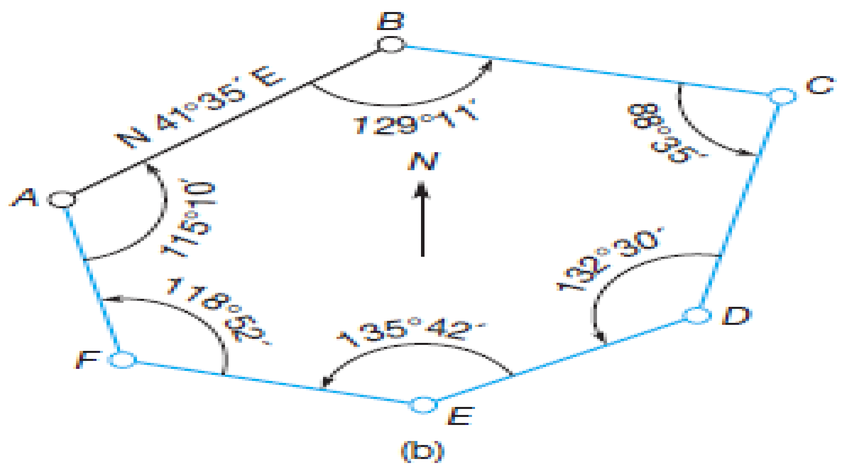
KINDS OF HORIZONTAL ANGLES

<u>The kinds of horizontal angles most commonly observed in surveying are:</u>

(1) Interior angles,
(2) Angles to the right, and
(3) Deflection angles.

sum of all interior angles in any polygon must equal: (n-2)*180°, n: no. of angle





Closed polygon. (a) Clockwise interior angles (angles to the right).

(b) Counterclockwise interior angles (angles to the left).

- **Exterior angles, located outside a closed polygon, explement interior angles. (explement: the quantity by which an angle or an arc falls short of 360° or a circle).**

- **The advantage to be gained by observing them is their use as another check, since the sum of the interior and exterior angles at any station must total 360°.**

- **Angles to the right are measured clockwise from the rear to the forward station.**

- **Most data collectors require that angles to the right be observed in the field.**

- **Angles to the left, turned counterclockwise from the rear station**

Angles to the right can be either interior or exterior angles of a closed polygon traverse.

Whether the angle is an interior or exterior angle depends on the direction the instrument proceeds around the traverse.

If the direction around the traverse is counterclockwise, then the angles to the right will be interior angles.

However, if the instrument proceeds clockwise around the traverse, then exterior angles will be observed.

If this is the case, the sum of the exterior angles for a closed-polygon traverse will be $(n+2)*180°$ Analysis of a simple sketch should make these observations clear.

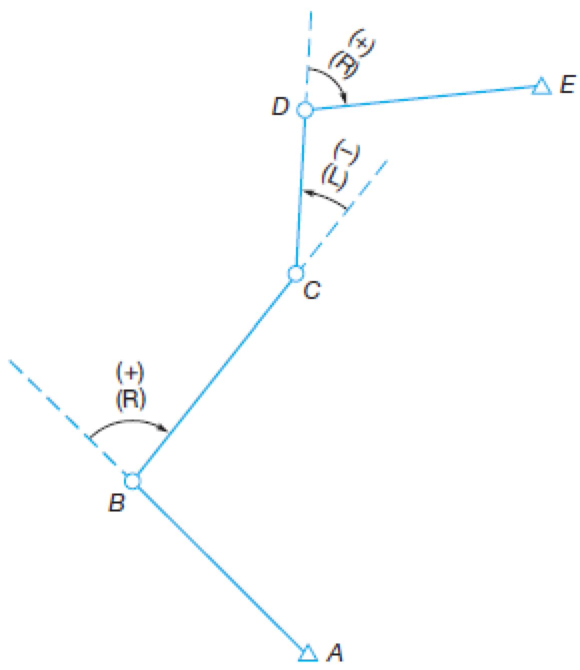
Deflection angles

Deflection angles are observed from an extension of the back line to the forward station. They are used principally on the long linear alignments of route surveys.

- Deflection angles may be observed to the right (clockwise) or to the left (counterclockwise) depending on the direction of the route.

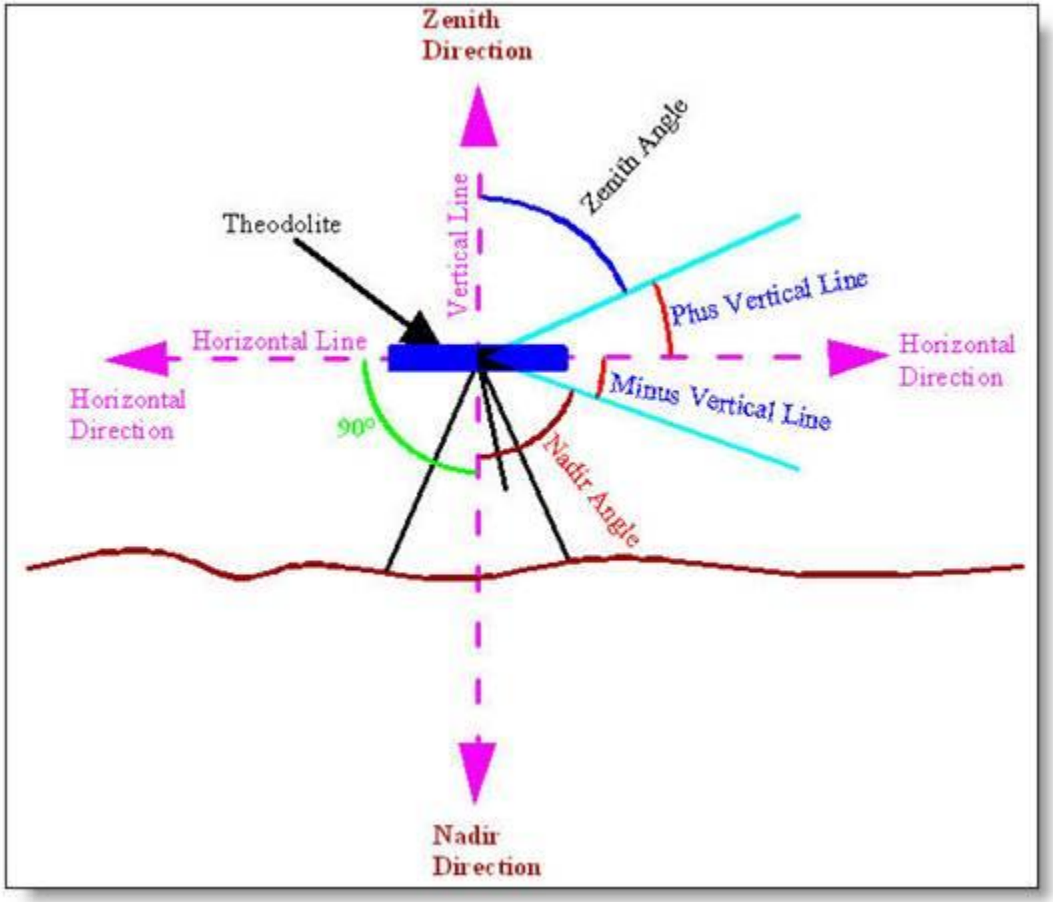
- Clockwise angles are considered plus, and counterclockwise ones minus, as shown in the figure.

- Deflection angles are always smaller than 180° and appending an R or L to the numerical value identifies the direction of turning.



Vertical Angles are referenced to:

- The horizon by plus (up) or minus (down) angles.
- The zenith: directly above the observer.
- The nadir: directly below the observer.



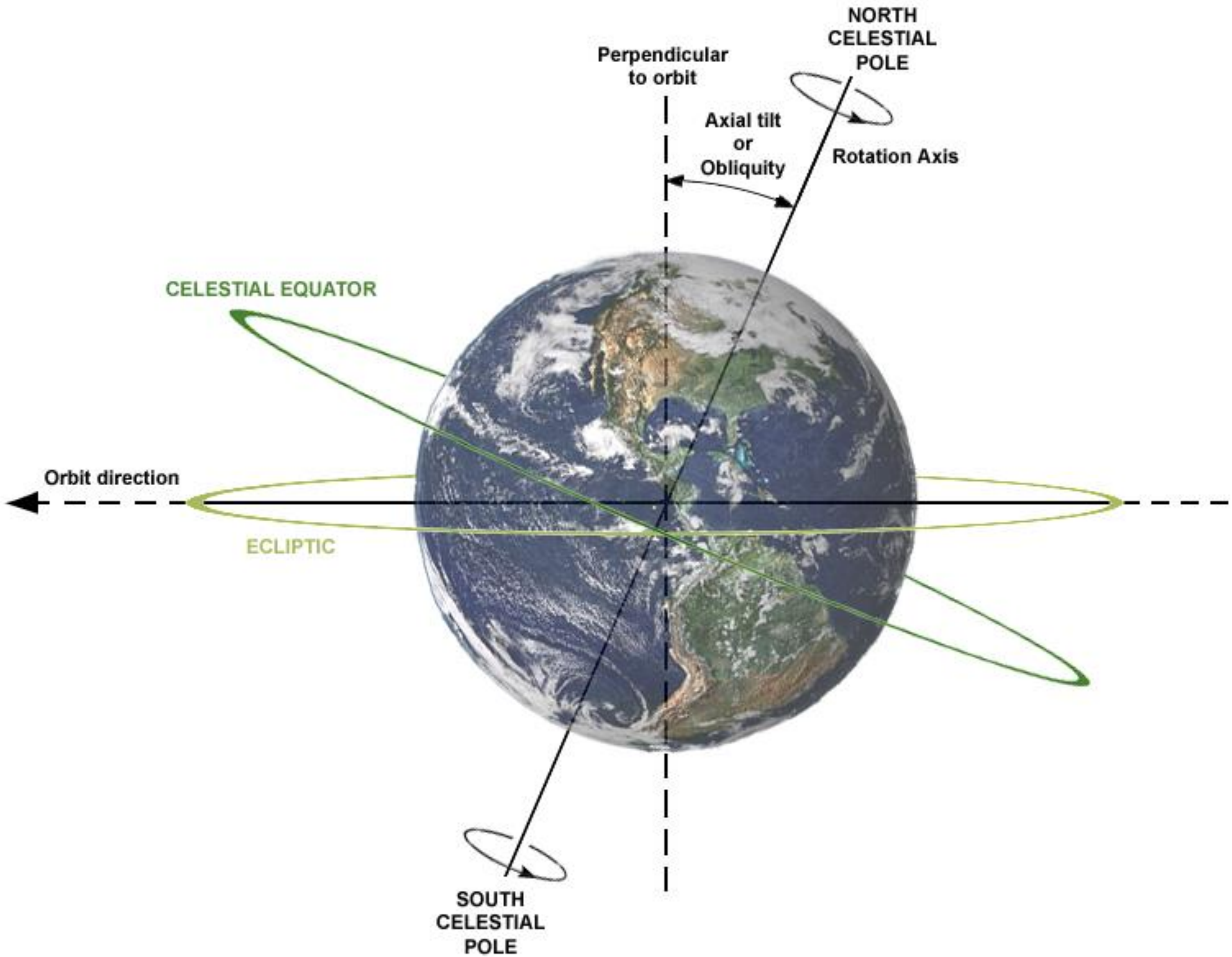
DIRECTION OF A LINE

The direction of a line is defined by the horizontal angle between the line and an arbitrarily chosen reference line called a *meridian*.

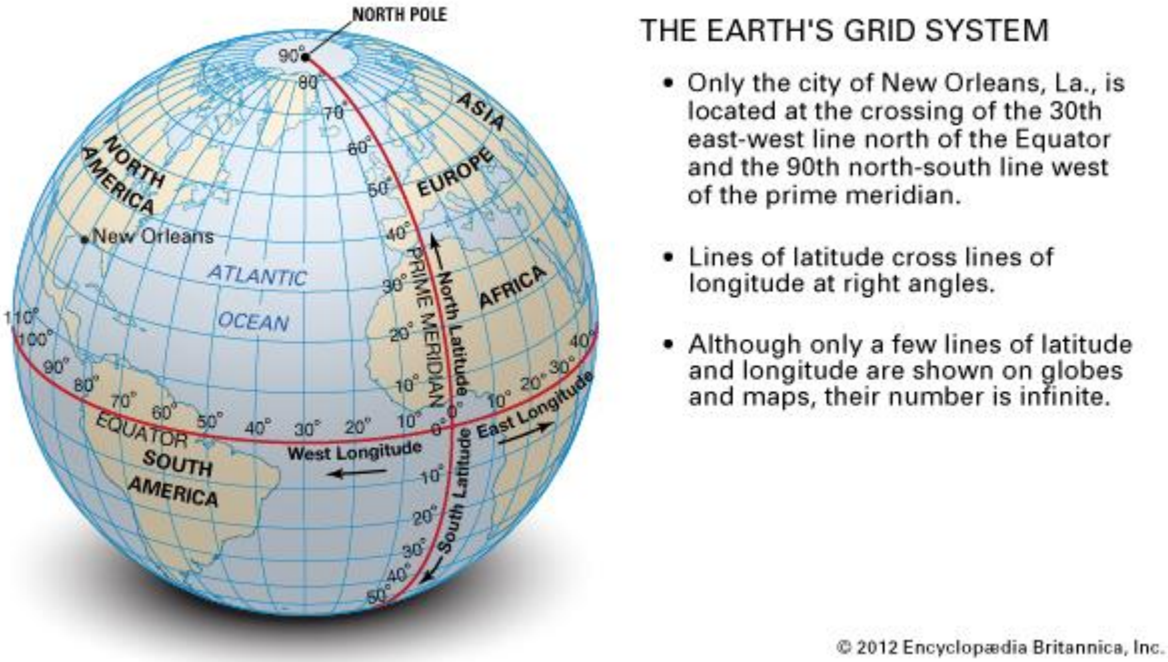
Different meridians are used for specifying directions including:

(a) geodetic (also often called true),
(b) astronomic,
(c) magnetic,
(d) grid,
(e) record, and
(f) assumed.

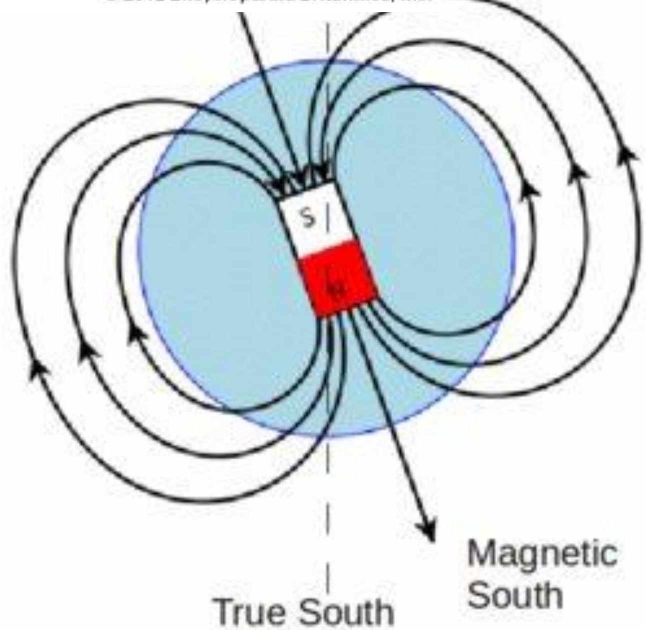
- **The geodetic meridian** is the north-south reference line that passes through a mean position of the Earth's geographic poles
- Wobbling (changing) of the Earth's rotational axis, causes the position of the Earth's geographic poles to vary with time.
- **Astronomic meridian** is the north-south reference line that passes through the instantaneous position of the Earth's geographic poles.
- Geodetic and astronomic meridians are very nearly the same, and the former can be computed from the latter by making small corrections
- **A magnetic meridian** is defined by a freely suspended magnetic needle that is only influenced by the Earth's magnetic field.
- Surveys based on a state or other plane coordinate system employ a **grid meridian for reference**. Grid north is the direction of geodetic north for a selected central meridian and held parallel to it over the entire area covered by a plane coordinate system


NORTH
CELESTIAL
POLE
Perpendicular
to orbit
Axial tilt
or
Obliquity
Rotation Axis
CELESTIAL EQUATOR
Orbit direction
ECLIPTIC
SOUTH
CELESTIAL
POLE

Grid



Magnetic



- In boundary surveys, the term **record meridian** refers to directional references quoted in the recorded documents from a previous survey of a particular parcel of land.

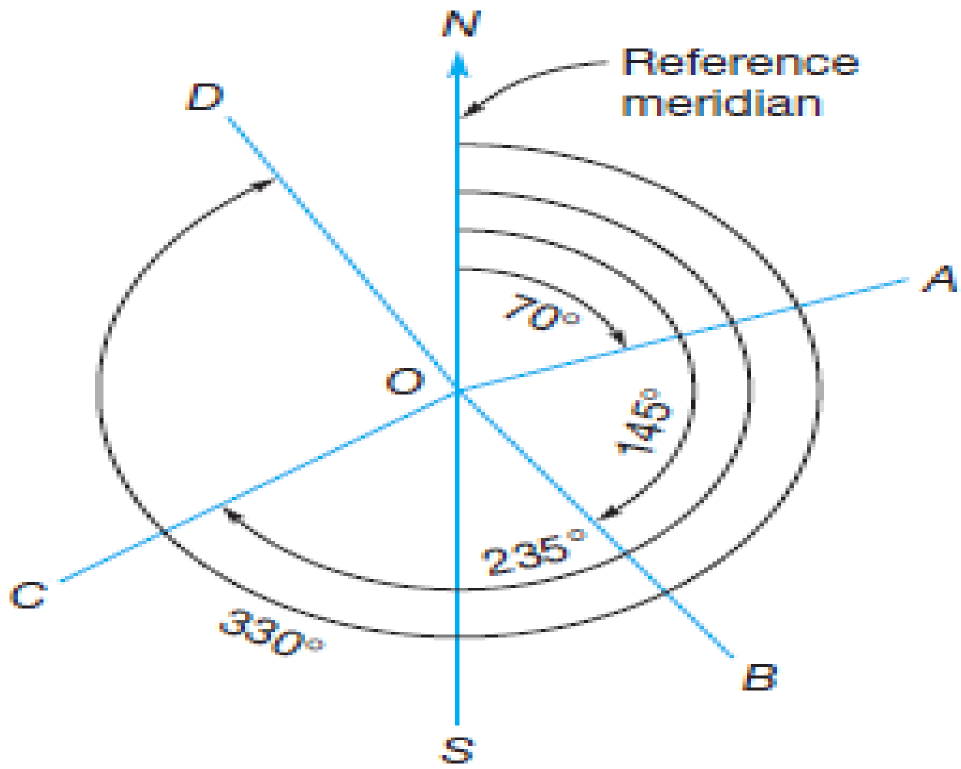
- An **assumed meridian** can be established by merely assigning any arbitrary direction—for example, taking a certain street line to be north. The directions of all other lines are then found in relation to it.

AZIMUTHS

- Azimuths are horizontal angles observed clockwise from any reference meridian. In plane surveying, azimuths are generally observed from north. Range: 0 - 360°

- A line's forward direction can be given by its forward azimuth, and its reverse direction by its back azimuth. In plane surveying, forward azimuths are converted to back azimuths, and vice versa, by adding or subtracting 180°.

AZIMUTHS

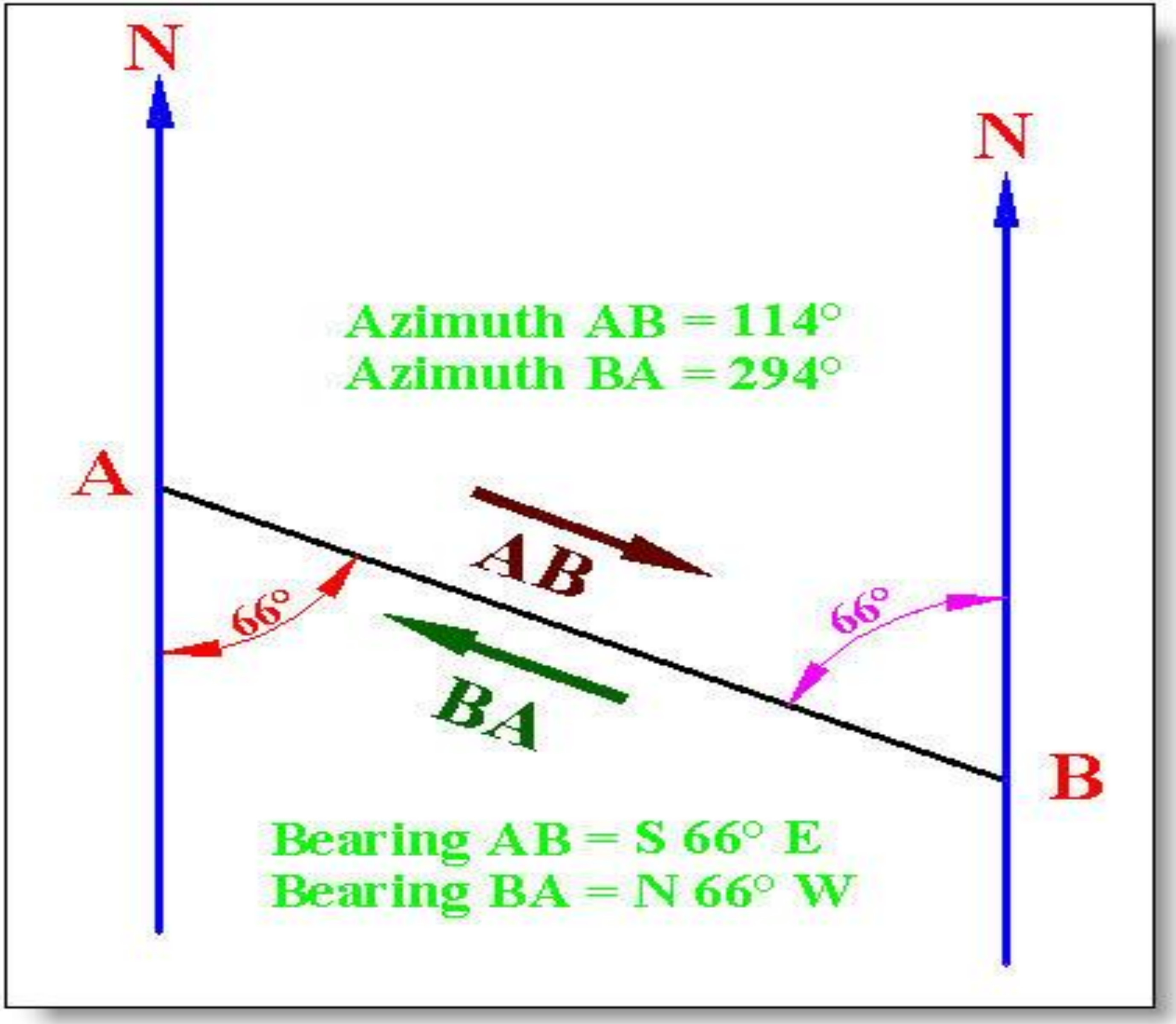


For example,

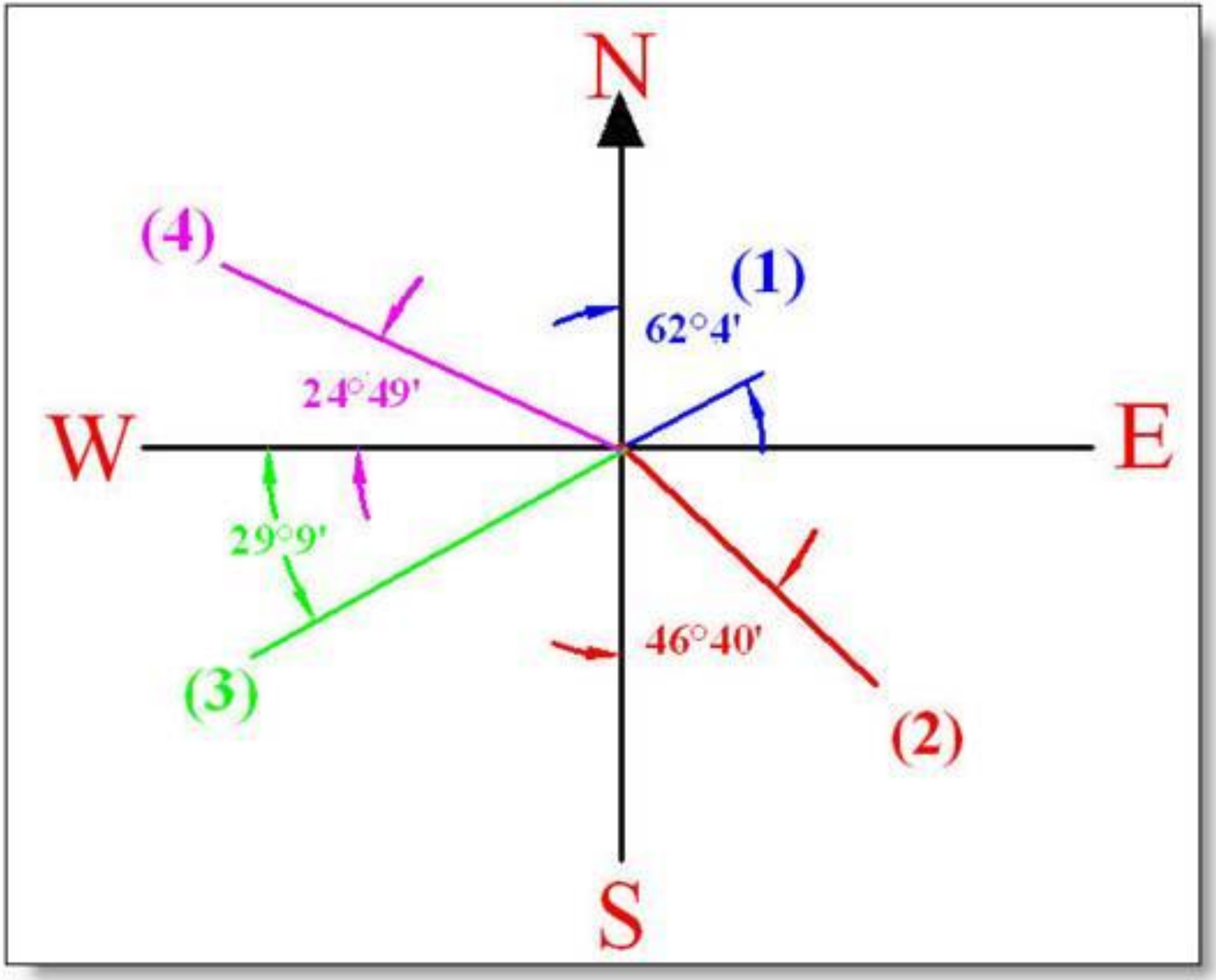
if the azimuth of OA is 70°, the azimuth of AO is 70°+ 180°=250°

If the azimuth of OD is 330°, the azimuth of DO is 330° - 180° = 150°.

To reverse Azimuth



Practice: Calculate the Azimuths of lines 1-4.



BEARINGS

Bearings are another system for designating directions of lines.

The bearing of a line is defined as the acute horizontal angle between a reference meridian and the line.

The angle is observed from either the north or south toward the east or west, to give a reading smaller than 90°.

The letter N or S preceding the angle, and E or W following it shows the proper quadrant.

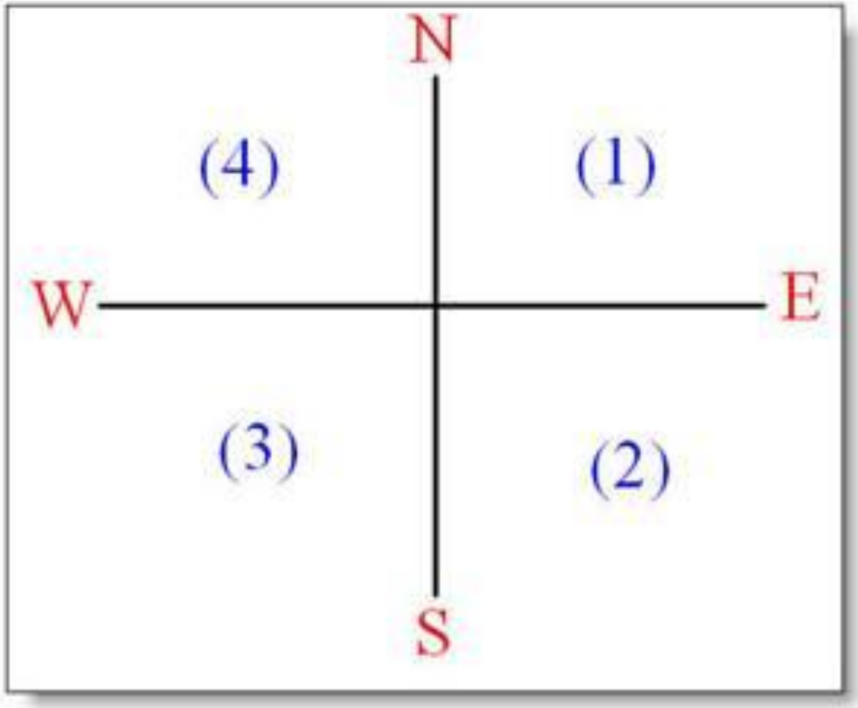
Bearing: Acute angle between N-S meridian and the line measured clockwise or counterclockwise.

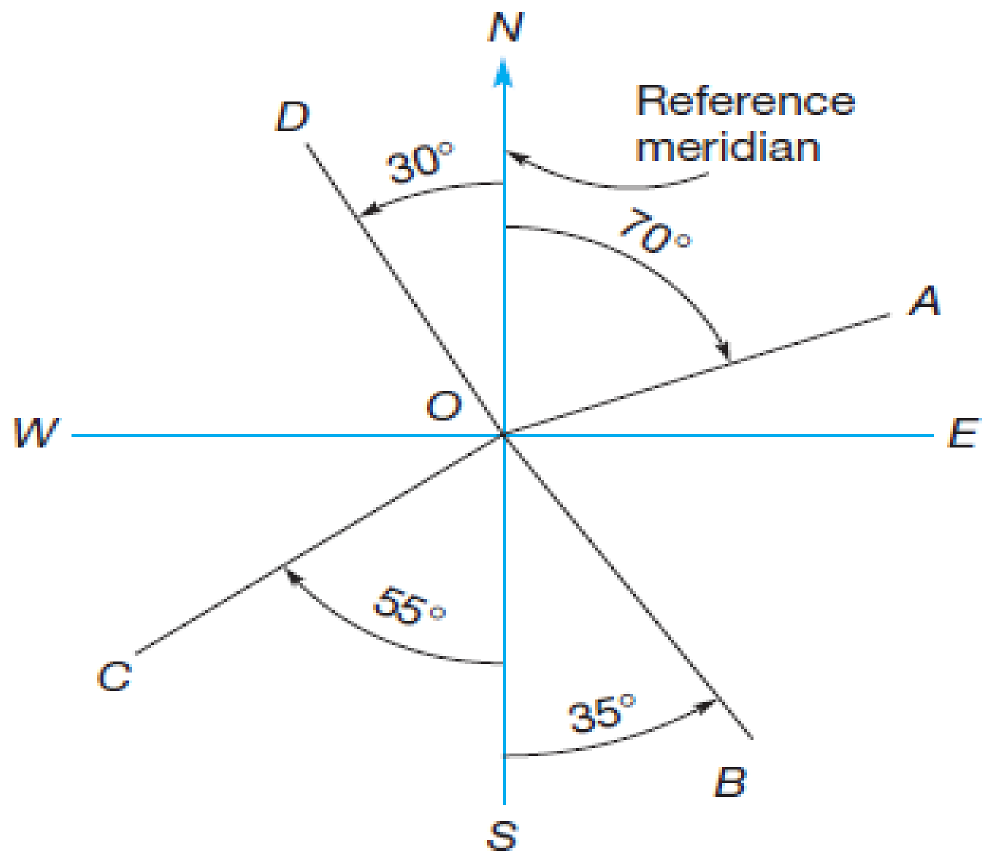
(1) N # # # E

(2) S # # # E

(3) S # # # W

(4) N # # # W





An example is N80°E all bearings in quadrant *NOE* are measured clockwise from the meridian.

Thus the bearing of line *OA* is N70°E.

All bearings in quadrant *SOE* are counterclockwise from the meridian, so *OB* is S35°E. Similarly, the bearing of *OC* is S55°W and that of *OD*, N30°W

When lines are in the cardinal directions, the bearings should be listed as "Due North," "Due East," "Due South," or "Due West."

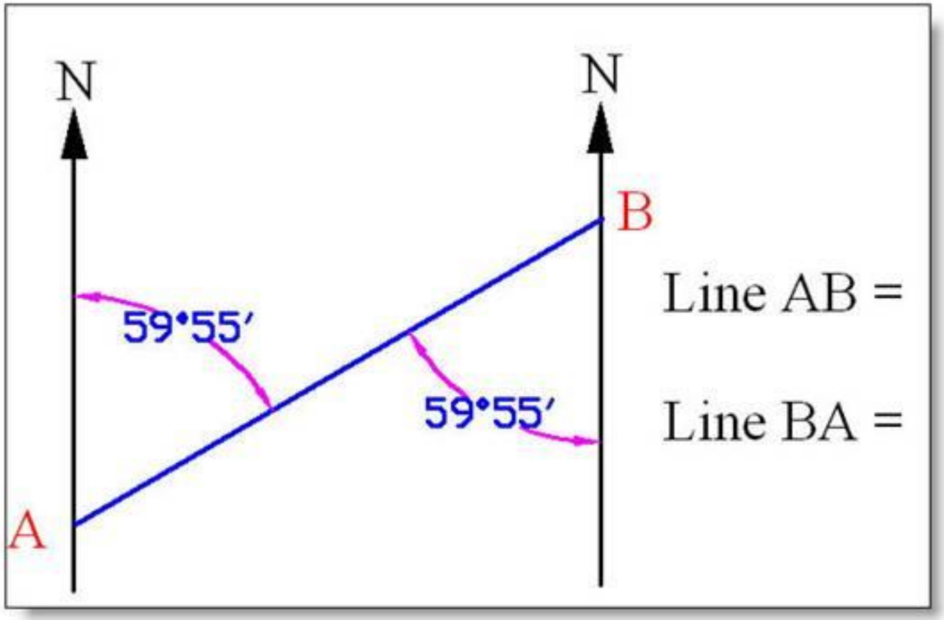
Back bearings should have the same numerical values as forward bearings but opposite letters. Thus if bearing *AB* is N44°E, bearing *BA* is S44°W.

To reverse <u>Bearing</u>:
Reverse direction letters

<u>AB BA</u>

N → S
S → N
E → W
W → E

and angles stay as is.



Practice: Calculate the Bearings of lines 5-8.

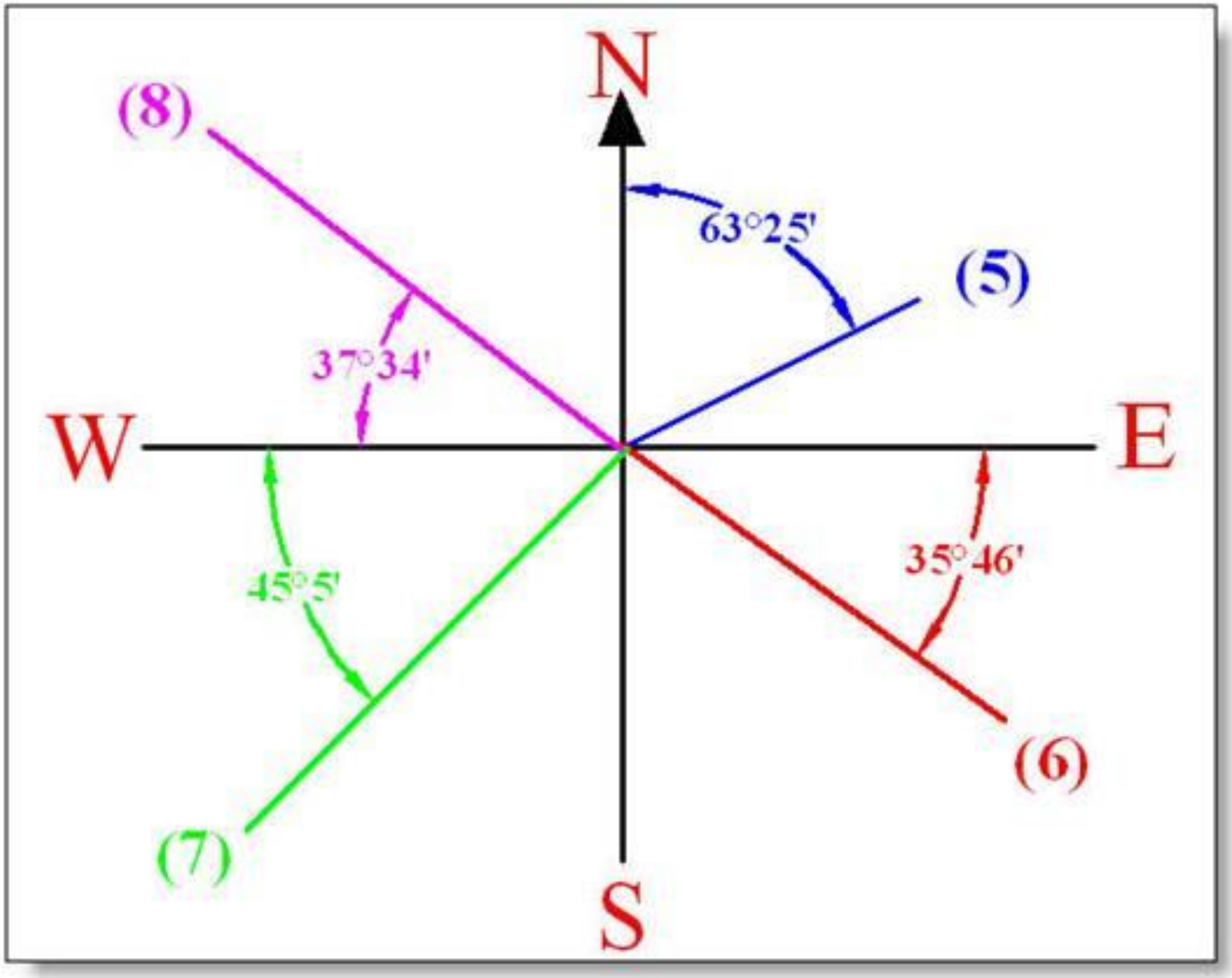


Table 7.1 Comparison of Azimuths and Bearings

Azimuths	Bearings
Vary from 0 to 360°	Vary from 0 to 90°
Require only a numerical value	Require two letters and a numerical value
May be geodetic, astronomic, magnetic, grid, assumed, forward or back	Same as azimuths
Are measured clockwise only	Are measured clockwise and counterclockwise
Are measured either from north only, or from south only on a particular survey	Are measured from north and south

Quadrant	Formulas for computing bearing angles from azimuths
I (NE)	Bearing = Azimuth
II (SE)	Bearing = 180° − Azimuth
III (SW)	Bearing = Azimuth − 180°
IV (NW)	Bearing = 360° − Azimuth

Example directions for lines in the four quadrants (azimuths from north)

Azimuth	Bearing
54°	N54°E
112°	S68°E
231°	S51°W
345°	N15°W

Example 7.1

The azimuth of a boundary line is 128°13′46″. Convert this to a bearing.

Solution

The azimuth places the line in the southeast quadrant. Thus, the bearing angle is

$$180° - 128°13'46'' = 51°46'14''$$

and the equivalent bearing is S51°46′14″E.

Example 7.2

The first course of a boundary survey is written as N37°13′W. What is its equivalent azimuth?

Solution

Since the bearing is in the northwest quadrant, the azimuth is

$$360° - 37°13' = 322°47'.$$

COMPUTING AZIMUTHS

Most types of surveys, but especially those that employ traversing, require computation of azimuths (or bearings).

A traverse is a series of connected lines whose lengths and angles at the junction points have been observed.

Traverses have many uses:

1. **To survey the boundary lines of a piece of property, for example, a "closed-polygon" type traverse would normally be used.**

2. **A highway survey from one city to another would usually involve a traverse**

Regardless of the type used, it is necessary to compute the directions of its lines.

- Many surveyors prefer azimuths to bearings for directions of lines because they are easier to work with, especially when calculating traverses with computers.

- Also sines and cosines of azimuth angles provide correct algebraic signs for departures and latitudes

Azimuth calculations are best made with the aid of a sketch

- Traverse angles must be adjusted to the proper geometric total before azimuths are computed.
  - in a closed-polygon traverse, the sum of interior angles equals 180(n-2)
  - If the traverse angles fail to close, it should be adjusted prior to computing azimuths

Azimuth Computation

1: Check interior angles sum = (n - 2) 180°

2: Counterclockwise (recommended)

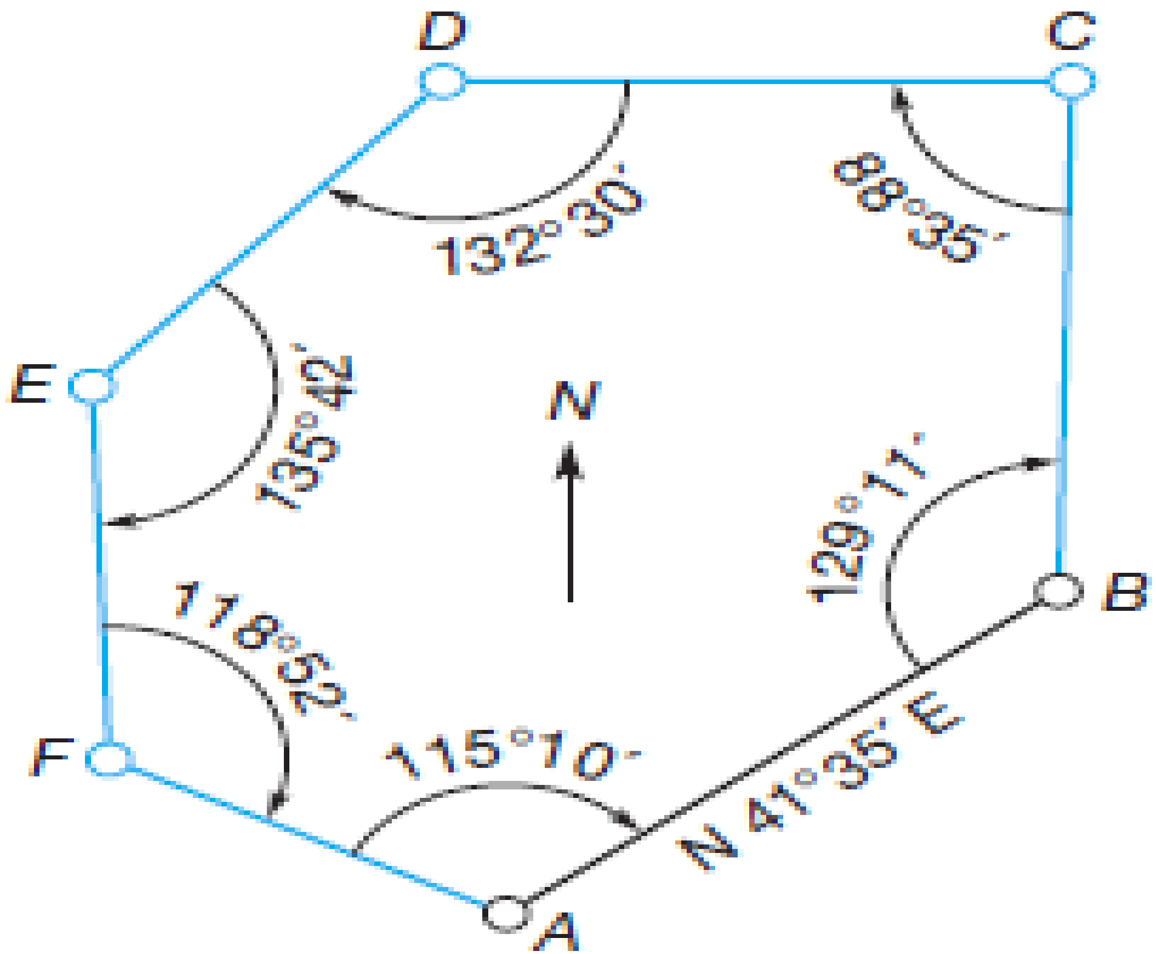
a- reverse Azimuth
b- add next interior angle
c- go to start and check

- 3: Clockwise

  a- find the Azimuth of the starting line (going clockwise)
  b- reverse Azimuth
  c- subtract interior angle
  d- go to start to check

- Note: you may need to add 360° to computations to facilitate subtraction.

CCW SOLUTION OF CLOSED TRAVERSE



CCW SOLUTION OF CLOSED TRAVERSE

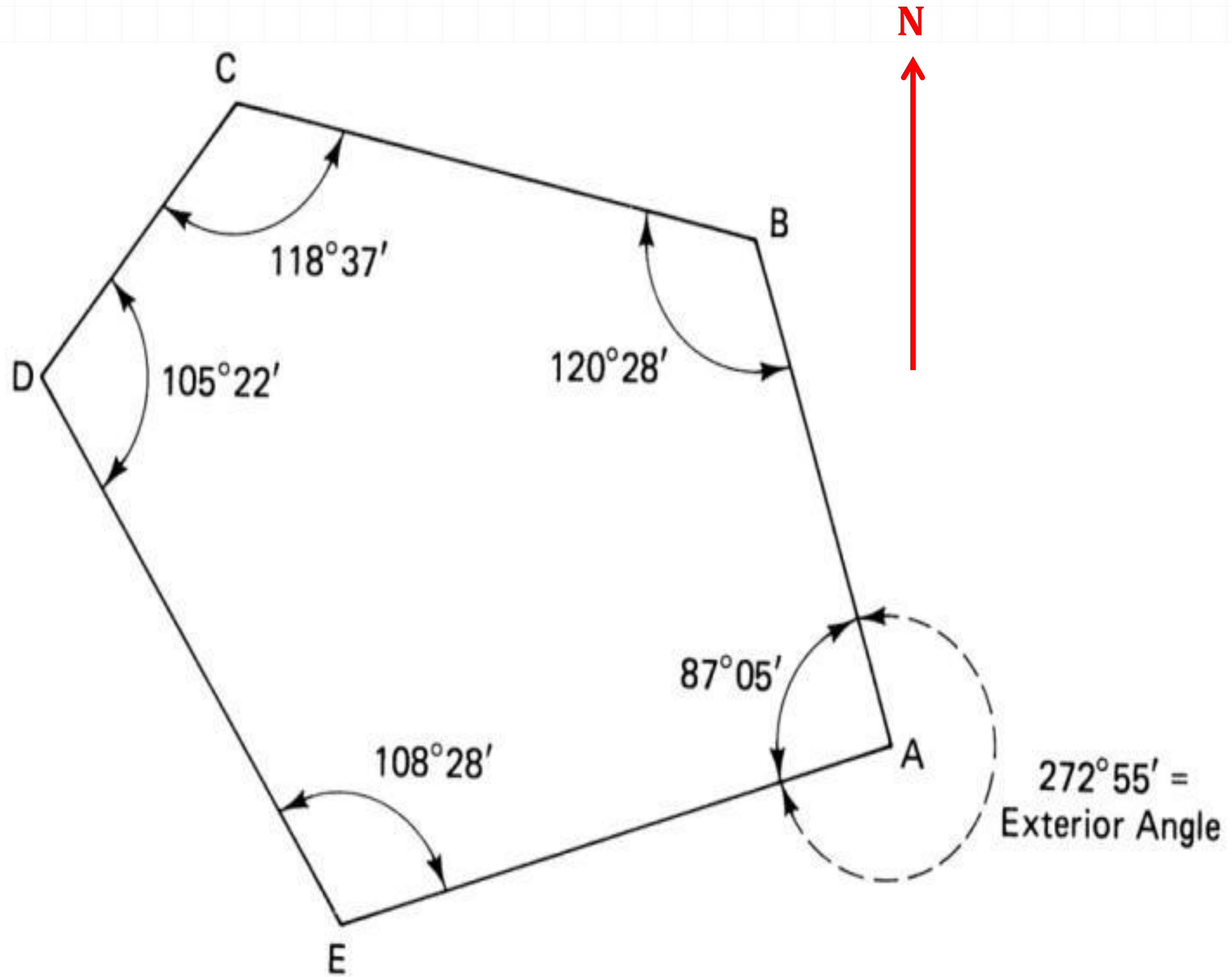
TABLE 7.2 COMPUTATION OF AZIMUTHS (FROM NORTH) FOR LINES OF FIGURE 7.2(a)

Angles to the Right [Figure 7.2(a)]

41°35′ = *AB*	211°51′ = *DE*
+180°00′	−180°00′
221°35′ = *BA*	31°51′ = *ED*
+129°11′	+135°42′
350°46′ = *BC*	167°33′ = *EF*
−180°00′	+180°00′
170°46′ = *CB*	347°33′ = *FE*
+88°35′	+118°52′
259°21′ = *CD*	466°25′ − *360° = 106°25′ = *FA*
−180°00′	−180°00′
79°21′ = *DC*	286°25′ = *AF*
+132°30′	+115°10′
211°51′ = *DE*	401°35′ − *360° = 41°35′ = *AB* ✔

*When a computed azimuth exceeds 360°, the correct azimuth is obtained by merely subtracting 360°.

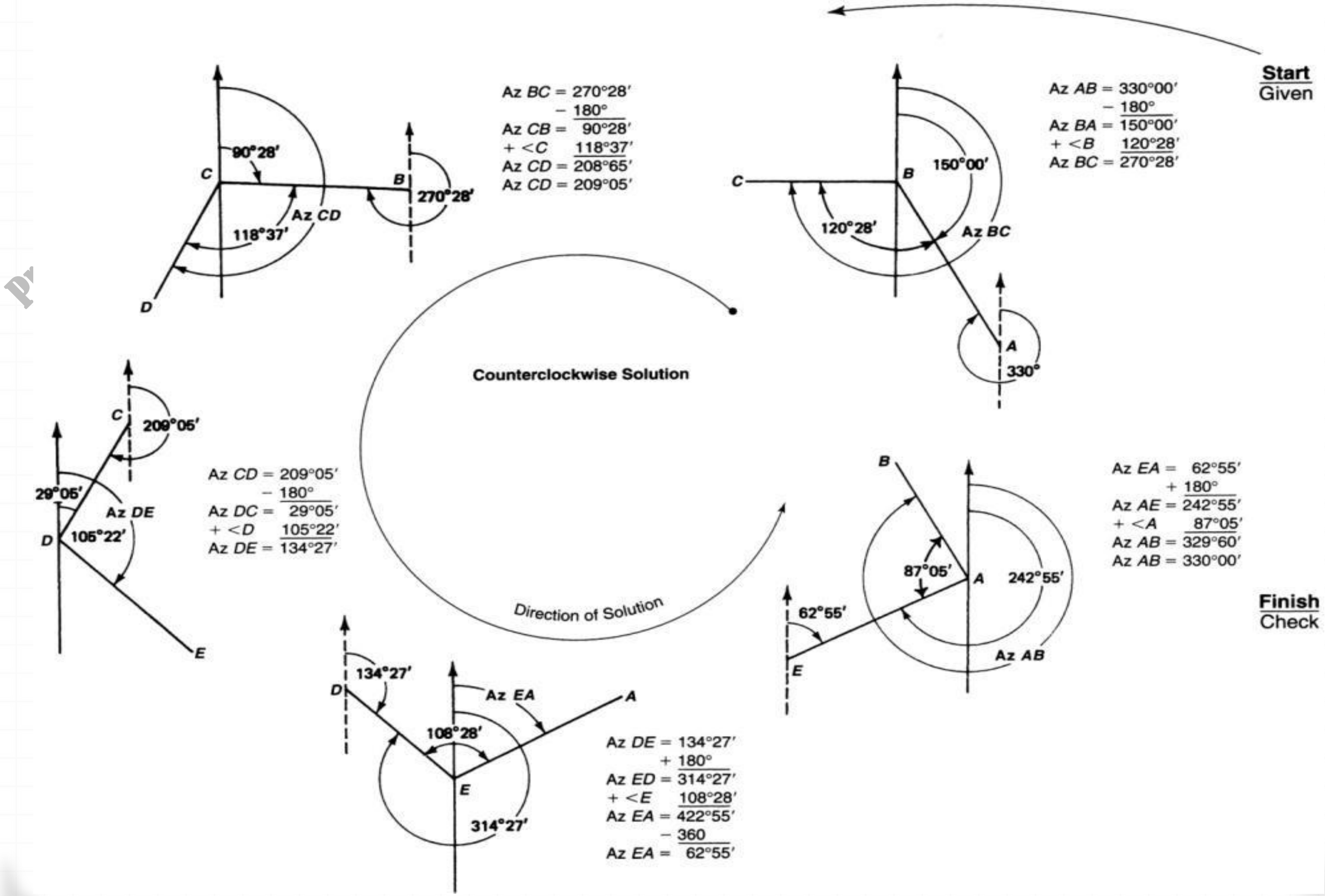
Another Example



A — 87°05′
B — 120°28′
C — 118°37′
D — 105°22′
E — 108°28′

538°120′
= 540°00′

Given: ……….All interior angles……………….Az AB=330 degree



CW SOLUTION OF CLOSED TRAVERSE

Course	Azimuth
AE	Az AE= Az AB – interior angle @ A = 330° - 87° 05' = 242° 55' 00"
EA	Az EA= Az AE -180° = 62° 55' 00" (back Az more than 180°) To enable subtraction of interior angle, add 360°→ Az EA= 62° 55'00"+360°= 422°55'00"
ED	Az ED= Az EA – Angle @ E =422°55'00"-108°28'00" =314°27'00"
DE	Az DE= Az ED -180° = 134°27'00"
DC	Az DC= Az DE-Angle @ D =134°27'00"-105°22'00"=29°05'00"
CD	Az CD= Az DC+180°= 209°05'00"
CB	Az CB= Az DC-Angle @ C =209°05'00"- 118°37'00"=90°28'00"
BC	Az BC= Az CB+180°=270°28°00"
BA	Az BA= Az BC-Angle @B =270°28'00"-120°28'00" =150°00"00"
AB	Az AB= Az BA+180°= 330°00'00" (Check)

Bearing Computations

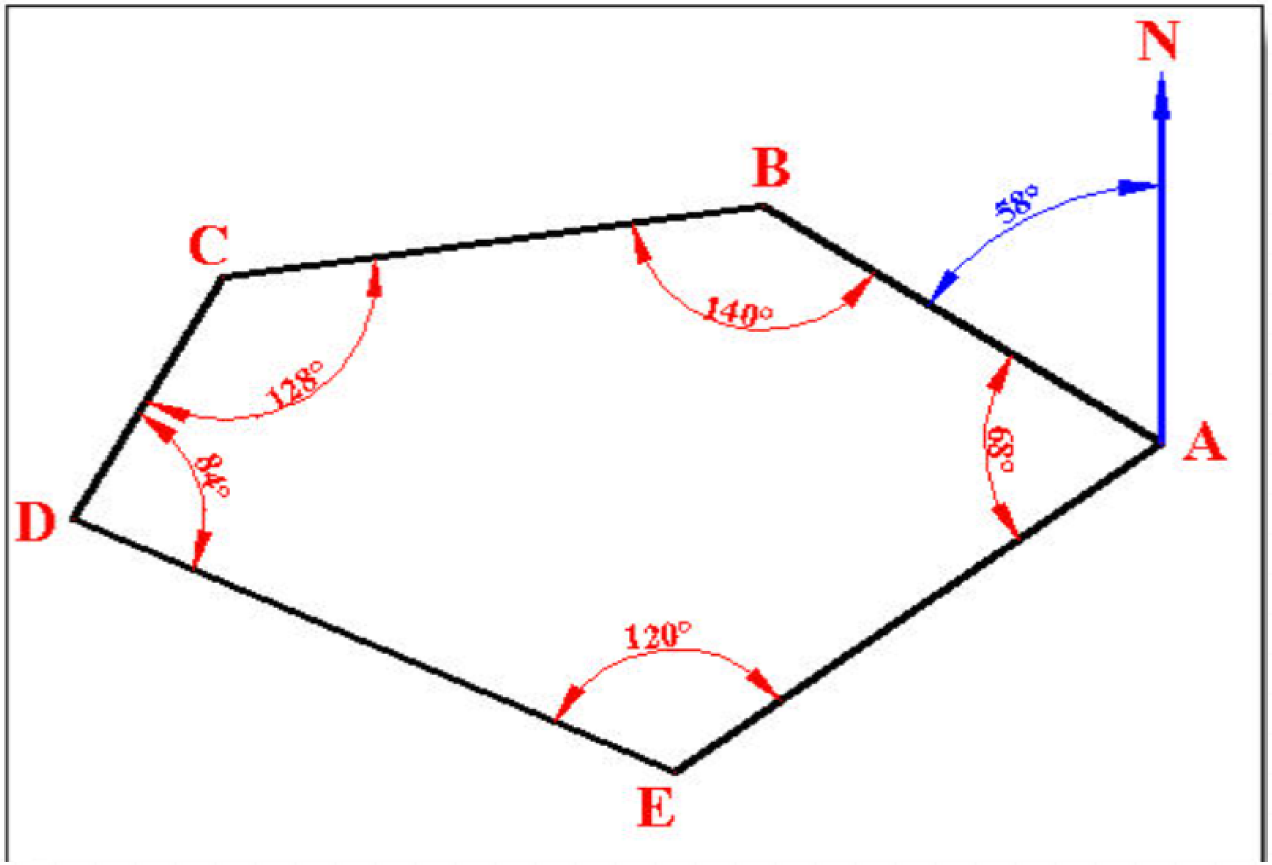
- The solution can proceed in **CW or CCW** manner.

- There is **no systematic method** of directly computing bearings, **each bearing computation will be regarded as a separate problem**,

- **Prepare a neat and a well labeled diagram sketch** showing the two traverse lines involved, with the meridian drawn through the angle station.

- On the sketch, show the **interior angle**, the **bearing angle** and **the required angle**. **Each bearing computation is regarded as a separate problem.**

Example: Find the Bearing /azimuth of all the lines of the traverse

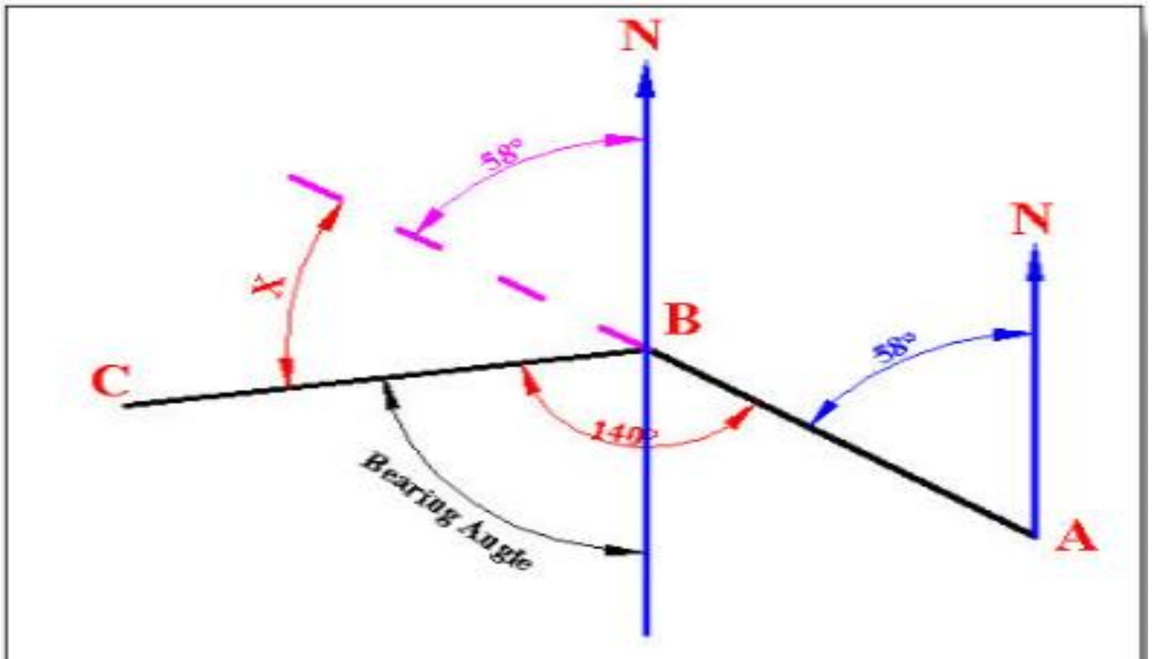


Solution / line Ab &BC

1- Line AB

N 58° W

2- Line BC



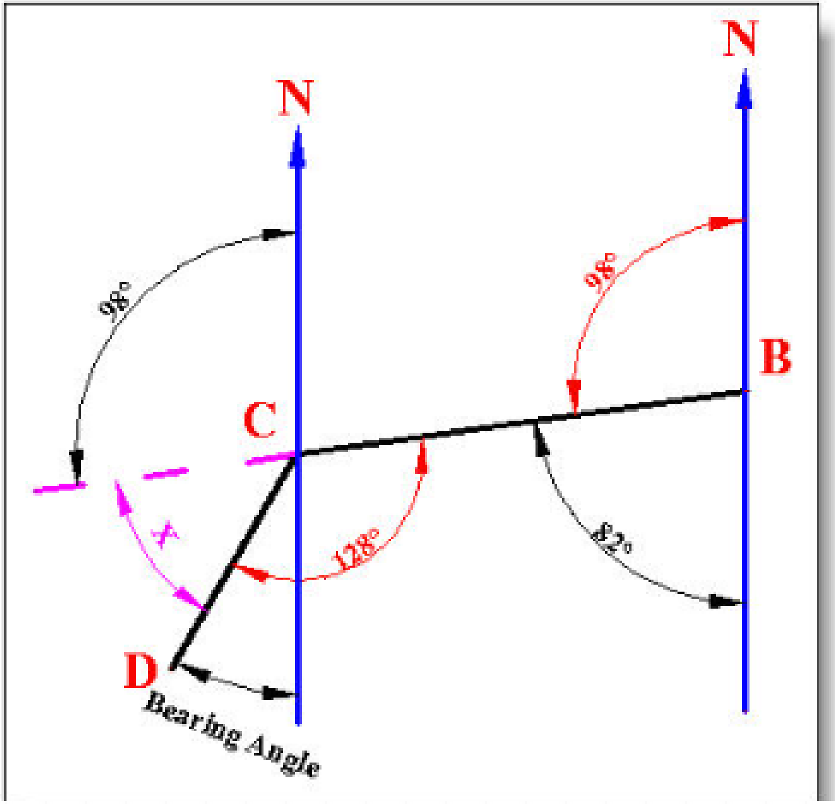
<X = 180° – 140° = 40°

Bearing Angle = 180° – 58° - 40° = 82°

BC = S 82° W

Solution/ line CD

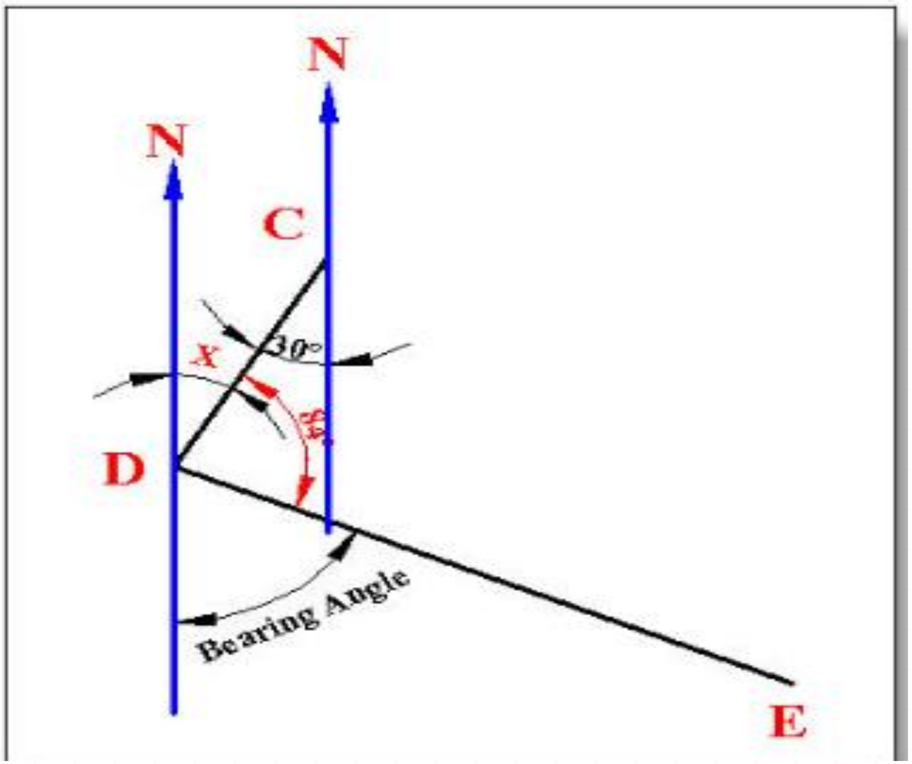
3- Line CD



<X = 180° – 128° = 52°
Bearing Angle = 180° – 98° - 52° = 30°
CD = S 30° W

Solution/ line DE

4- Line DE



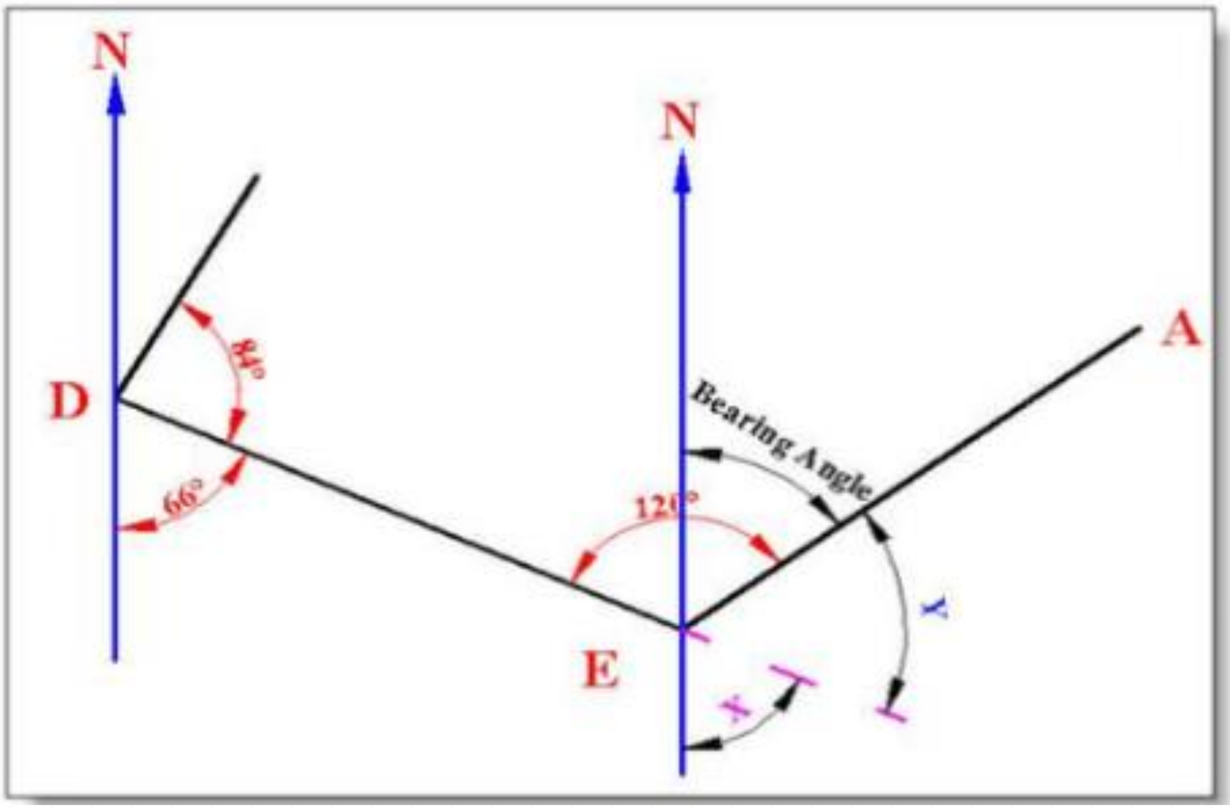
<X = 30° **Why?**

Bearing Angle = 180° – 84° - 30° = 66°

DE = S 66° E

Solution/ line EA

5- Line EA

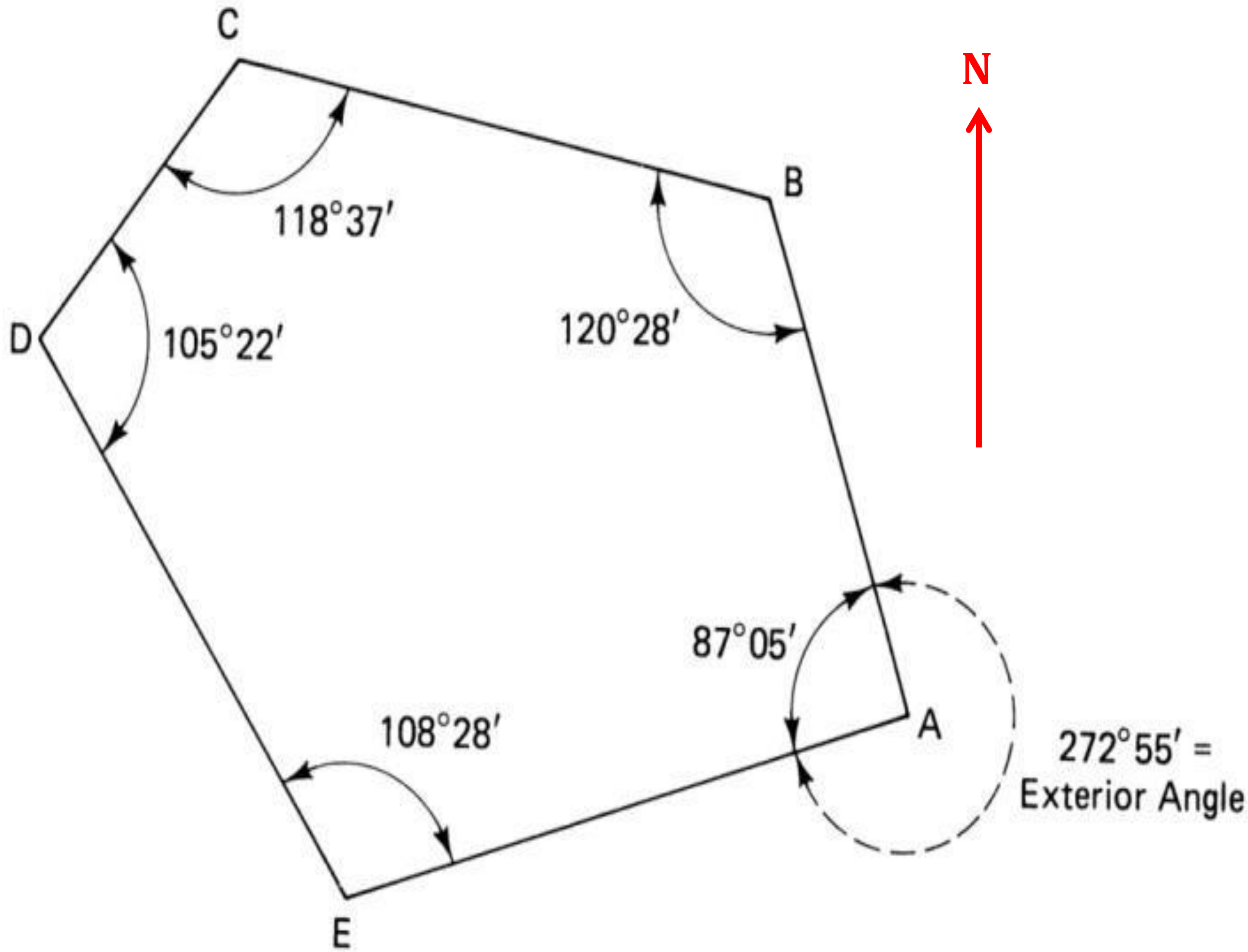


<X = 66° **Why?**
<Y = 180 - 120° = 60°
Bearing Angle = 180° – 60° - 66° = 54°
EA = N 54° E

A — 87°05′
B — 120°28′
C — 118°37′
D — 105°22′
E — 108°28′

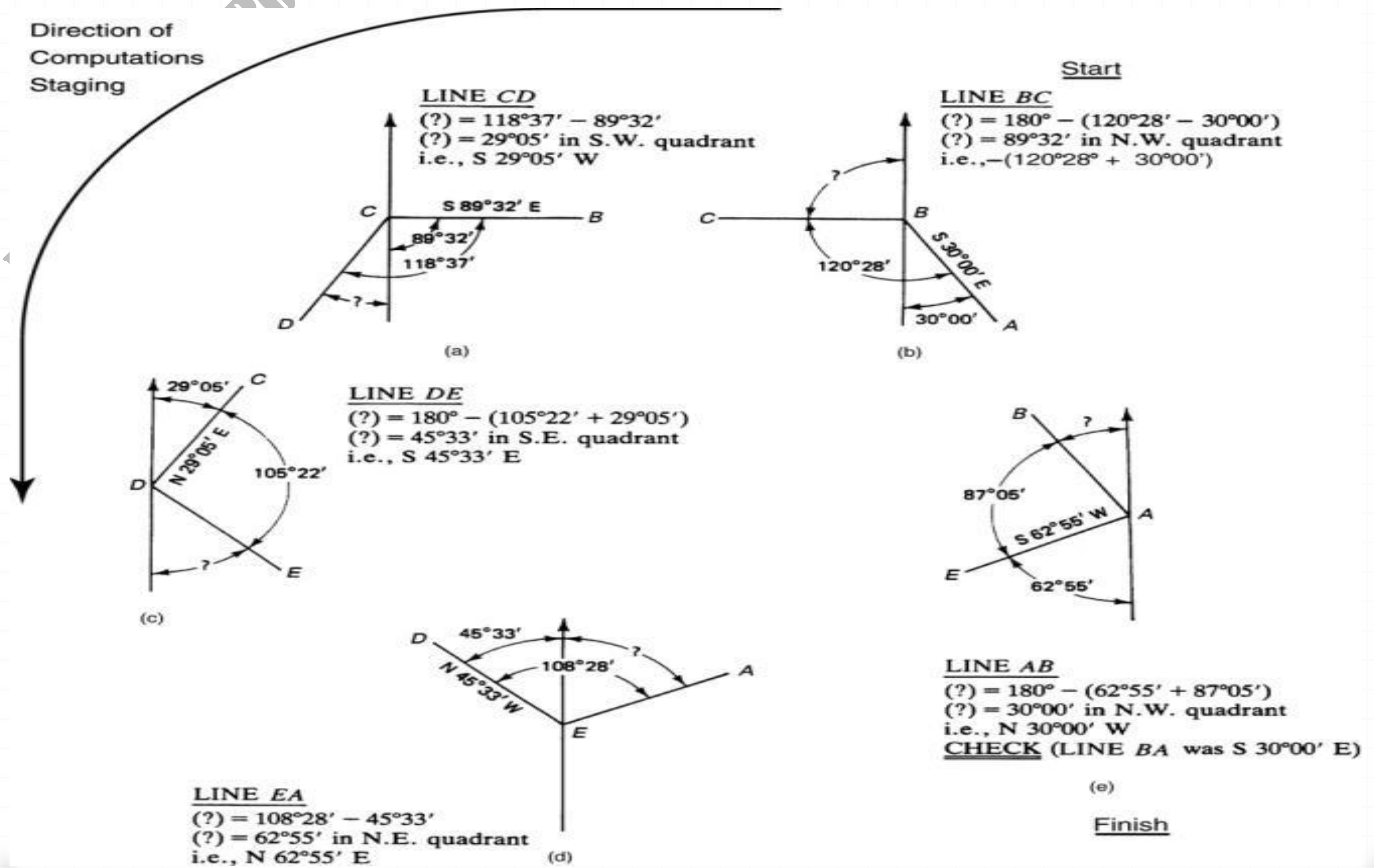
538°120′

= 540°00′



Bearing of course BA= S30°00’00’’E

Sketch for bearing Computation

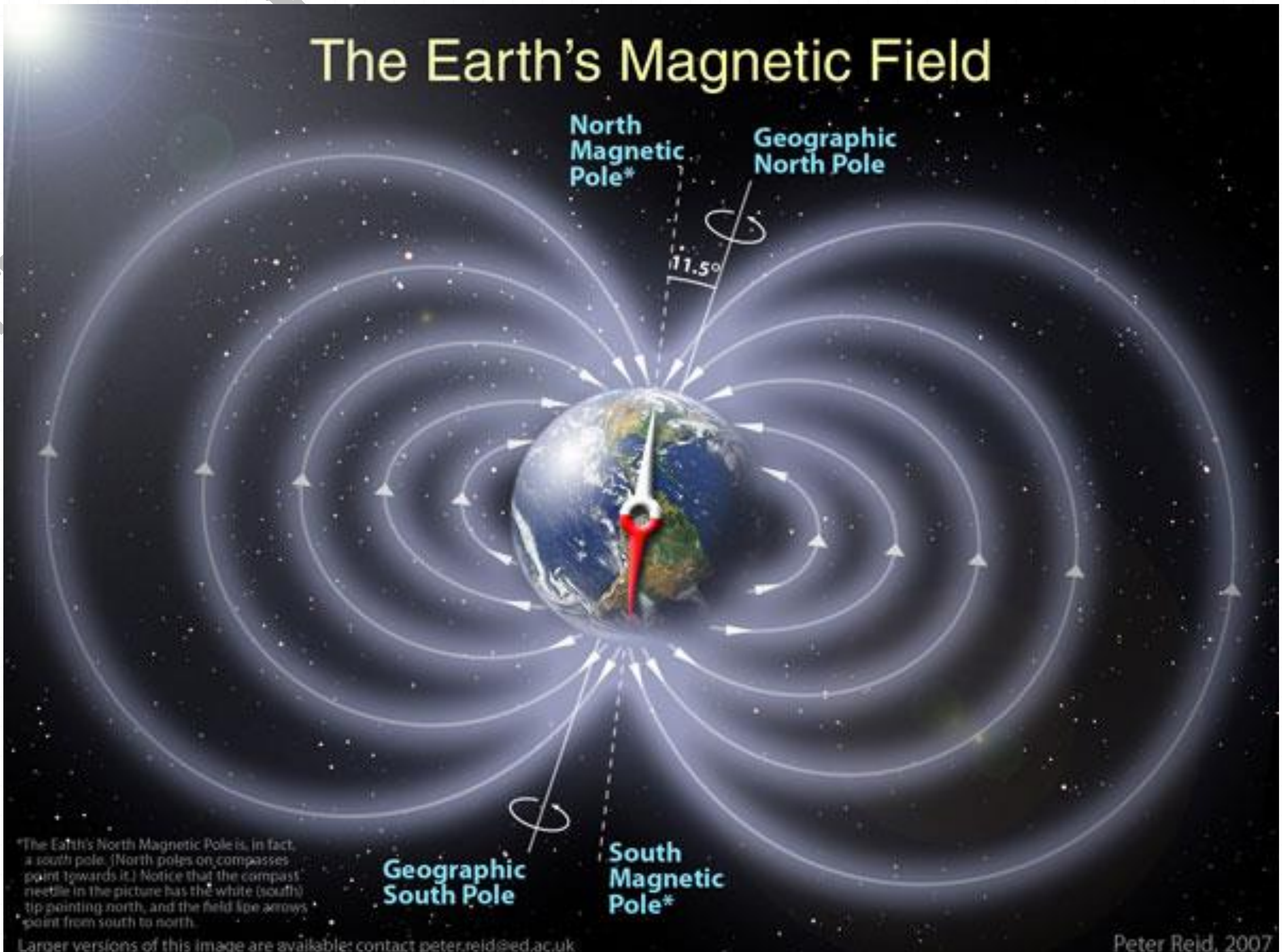


Magnetic Direction

- The **North Magnetic Pole** is the point on the surface of Earth's Northern Hemisphere at which the planet's magnetic field points vertically downwards (in other words, if a magnetic compass needle is allowed to rotate about a horizontal axis, it will point straight down).
- The North Magnetic Pole moves over time due to magnetic changes in the Earth's core.
- Its southern hemisphere counterpart is the South Magnetic Pole. Since the Earth's magnetic field is not exactly symmetrical, the North and South Magnetic Poles are not antipodal: i.e., a line drawn from one to the other does not pass through the geometric Center of the Earth.
- The direction of magnetic field lines are defined to emerge from the magnet's south pole and enter the magnet's north pole.

- **True North** (geodetic north) is the direction along the earth's surface towards the geographic North Pole.

- True geodetic north differs from magnetic north.



The Earth's Magnetic Field
North Magnetic Pole*
Geographic North Pole
11.5°
Geographic South Pole
South Magnetic Pole*
*The Earth's North Magnetic Pole is, in fact, a south pole. (North poles on compasses point towards it.) Notice that the compass needle in the picture has the white (south) tip pointing north, and the field line arrows point from south to north.
Larger versions of this image are available: contact peter.reid@ed.ac.uk
Peter Reid, 2007

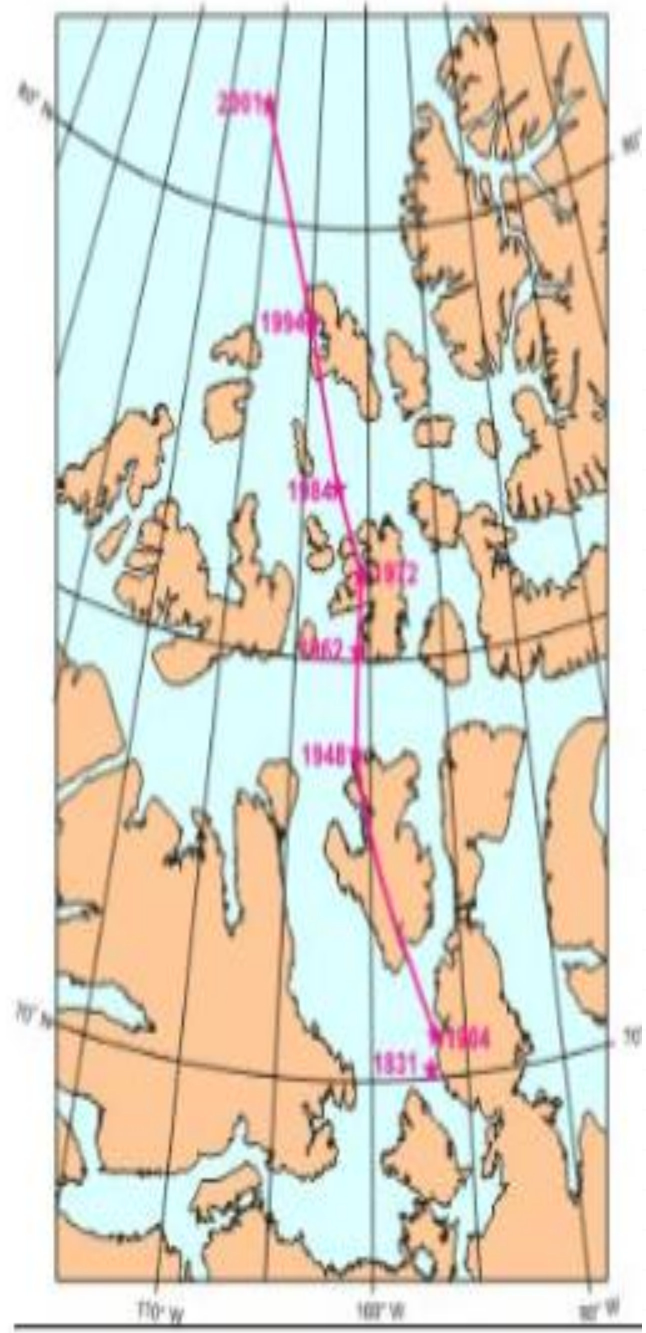
Magnetic North and Magnetic Declination

- The compass aligns itself to the local geomagnetic field, which varies in a complex manner over the Earth's surface, as well as over time.
- The local angular difference between magnetic north and true north is called the magnetic declination.
- Most map coordinate systems are based on true north, and magnetic declination is often shown on map legends so that the direction of true north can be determined from north as indicated by a compass.
- Many countries issue *isogonic charts*, usually every 5-10 years, on which lines are drawn (isogonic lines) that join points on the earth's surface that are experiencing equal annual changes in magnetic declination.
- *Due to uncertainties of determining magnetic declination, magnetic directions are not employed for any but the lowest order of surveys.*

Magnetic Declination for USA – 2010 epoch

Magnetic Direction

Movement of the magnetic North with time



Prof. TALEB AL-ROUSAN

Surveying

2104011365

Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University

Prof. TALEB AL-ROUSAN

Chapter 6
Traverse Surveys

Traverse Surveys

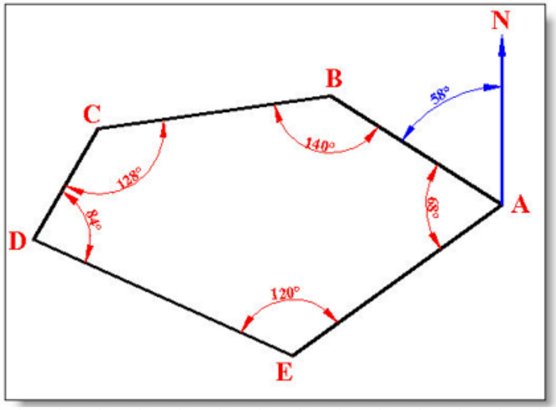
- **Traverse: is a <u>control survey</u> which is a series of established stations that are tied together by angles and distances.**

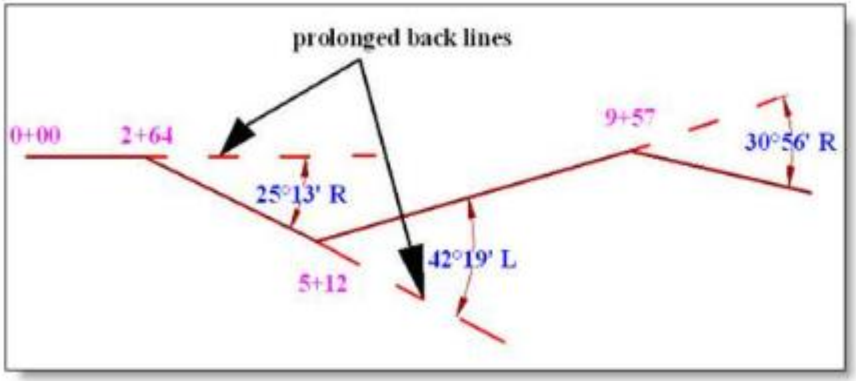
Uses: (1) Locate topographic details

(2) Layout engineering work

(3) Processing earth work.

- **Types:** closed traverse & open Traverse





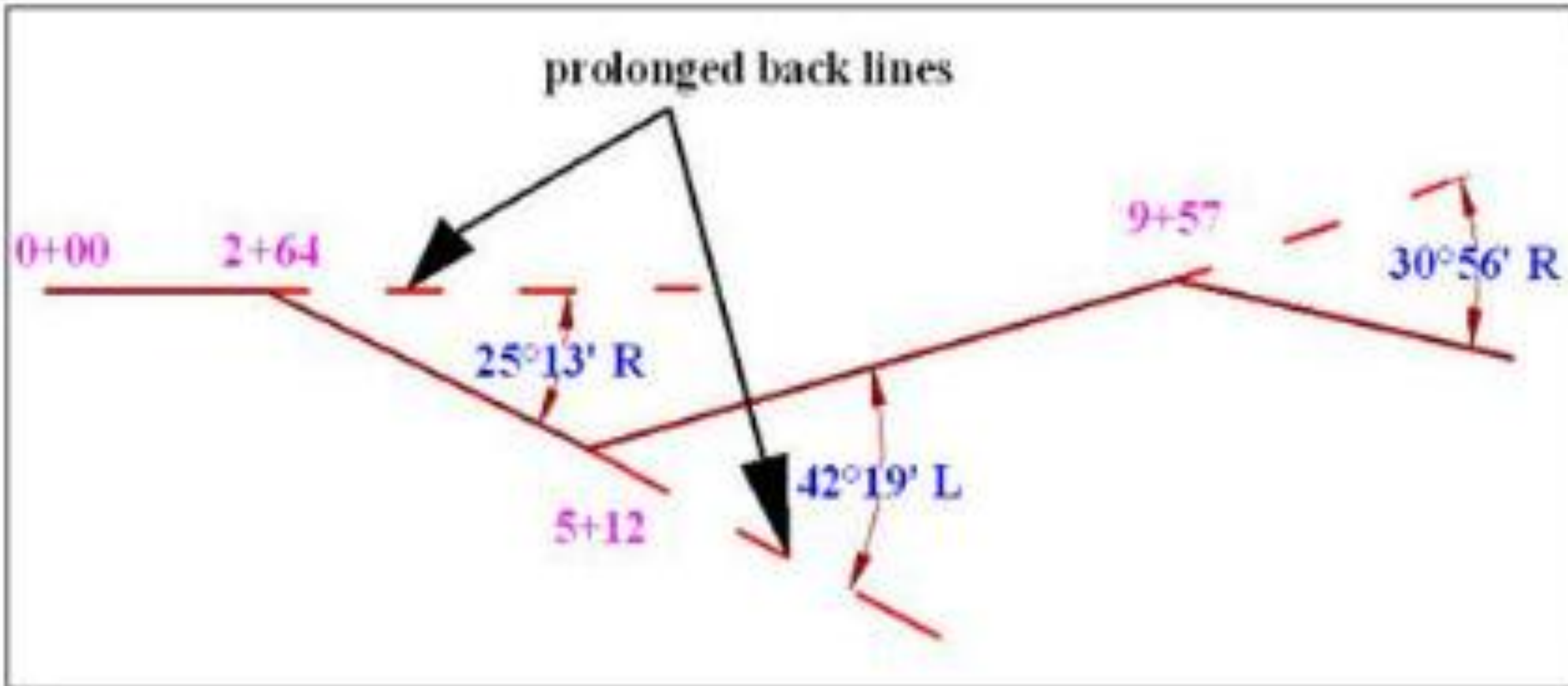
Open Traverses

- A series of measured straight lines and angles that do not geometrically close.
- No geometric verification of stations. So for field verifications:
  - Distances measured twice
  - Angles doubled

- When traverse stations can be tied to control monuments (BM's) or by using accurate GPS then verification is possible (it becomes Closed Travers).

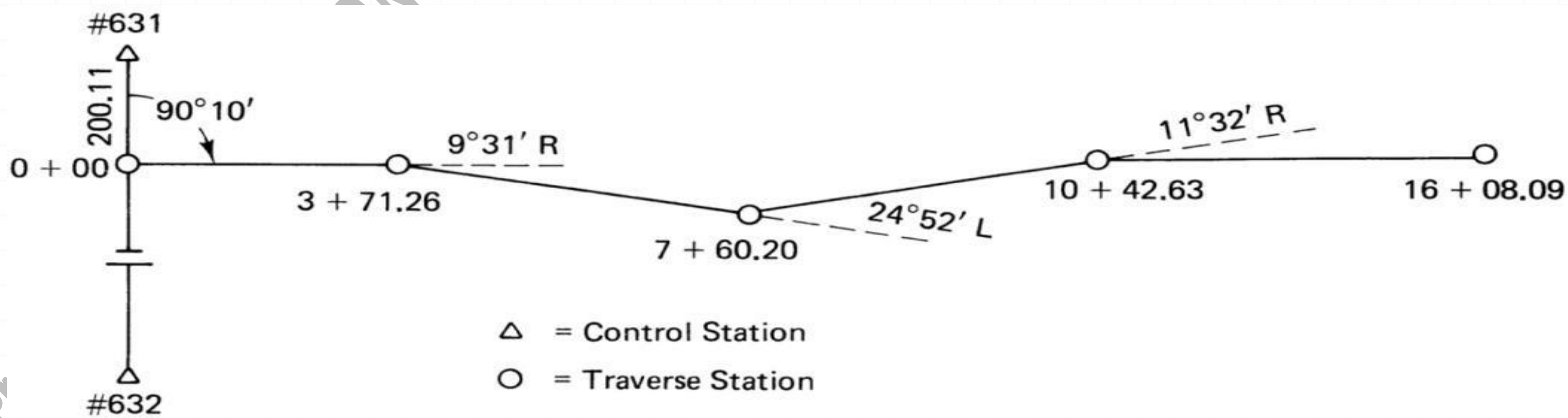
  - In route survey, open traverse stations can be verified by tying in the initial and terminal stations of a route survey to coordinate grid monuments whose positions are known.
  - In this case, the route survey becomes a closed traverse and is subject to geometric verification and analysis

Open Traverses



Field notes of the above drawing

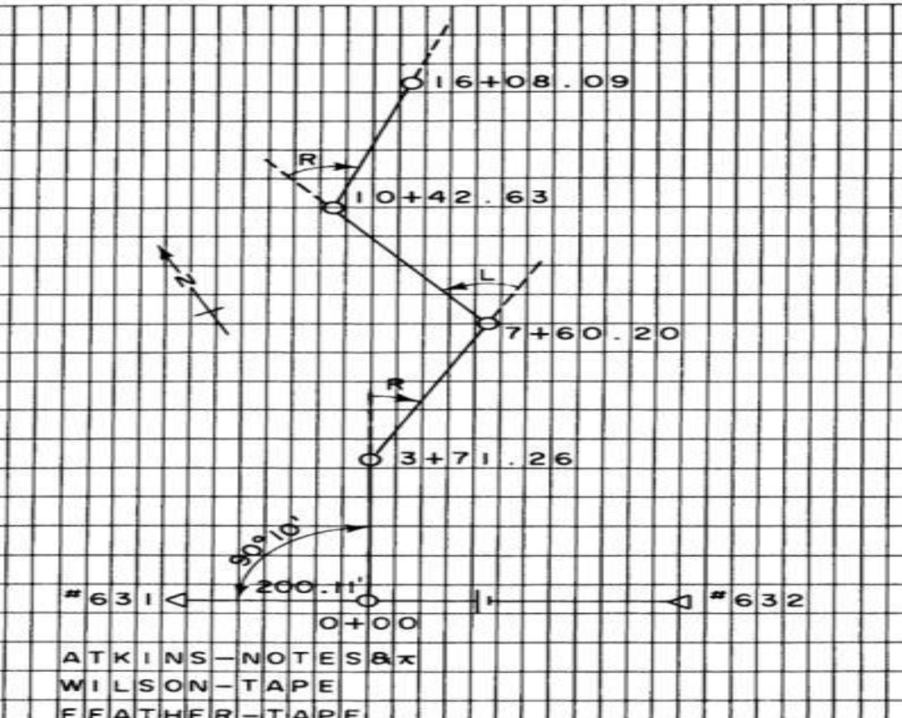
Station	Direct	Double	Mean	L/R
0+00	90° 00’	180° 00’	90° 00’	N 90° 00’ E
2+64	25° 12’	50° 26’	25° 13’	R
5+12	42° 20’	84° 38’	42° 19’	L
9+57	30° 56’	61° 52’	30° 56’	R
Chainages				
	Direct	Reverse	Mean	Corrected Chainage
				0+00
	264.3	264.5	264.4	2+64.4
	248.1	248.0	248.1	5+12.5
	445.4	445.8	445.6	9+58.1
	235.7	235.9	235.8	11+93.9



PRELIMINARY SURVEY FOR CLEAR LAKE
ACCESS ROAD—TOMLIN TOWNSHIP

Job CLEAR 68° F
Date MAY 18, 2005 Page 14

STATION	DIRECT	DOUBLE	MEAN	L/R	
0 + 00	90° 11'	180° 20'	90° 10'	SEE SKETCH.	
	DEFLECTION ANGLES				
3+71.26	9° 31'	19° 02'	9° 31'	R	
7+60.20	24° 51'	49° 44'	24° 52'	L	
10+42.63	11° 32'	23° 04'	11° 32'	R	
	CHAINAGES				
	DIRECT	REVERSE	MEAN		CHAINAGES
					0 + 00.00
	371.24	371.28	371.26		3 + 71.26
	388.93	~~388.80~~			
		388.95	388.94		7 + 60.20
	282.43	282.43	282.43		10 + 42.63
	565.44	565.49	565.46		16 + 08.09
			1608.09		



Closed Traverse

- Begins and ends at the same points (Loop traverse)
- Begins and ends at points of known position
- **Balancing Angles:**
- This is the first step in Traverse calculation
- Interior angle sum = (n - 2) 180
- Distribute error equally (recommended) , arbitrary or according to weights.
- Acceptable angular closure error is usually quite small (i.e., < 03')

Traverse Computations

Traverse computations include the following:

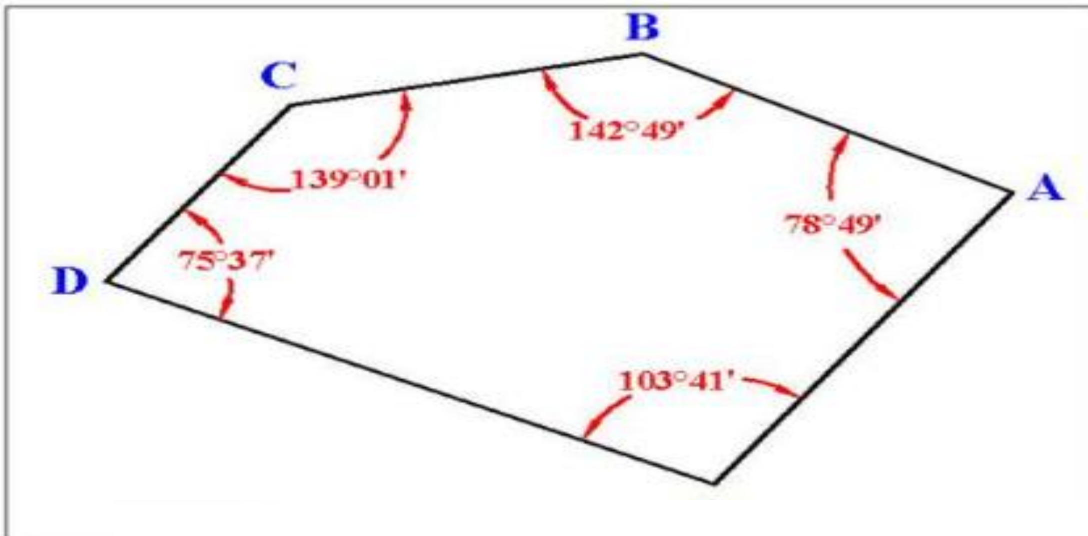
1. **Balancing field angles**
2. **Compute latitudes and departures**
3. **Compute traverse error**
4. **Balance latitudes and departures**
5. **Adjust original distances and directions**
6. **Compute coordinates of the traverse stations**
7. **Compute area enclosed by a closed traverse**

In modern practice these computations are routinely performed on computers and on total stations

Closed Traverse/ Example

Adjusting sum of angles according to weights

Balance the following traverse.



Point	Angle Value	Correction	Corrected Angle
A	78° 49'	(78°49' / 540°) * 03' = + 00° 00' 26"	78° 49' 26"
B	142° 49'	(142°49' / 540°) * 03' = + 00° 00' 48"	142° 49' 48"
C	139° 01'	+ 00° 00' 46"	139° 01' 46"
D	75° 37'	+ 00° 00' 25"	75° 37' 25"
E	103° 41'	+ 00° 00' 35"	103° 41' 35"
Total	539° 57'	+ 00° 03'	540° 00' 00"

Closed Traverse/ Example

Table 6.1 TWO METHODS OF ADJUSTING FIELD ANGLES

Station	Field Angle	Arbitrarily Balanced		Equally Balanced
A	101° 24′00″	101° 24′00″		101° 24′ 12″
B	149° 13′00″	149° 13′00″		149° 13′ 12″
C	80° 58′30″ →	80° 59′00″	30″	80° 58′ 42″
D	116° 19′00″	116° 19′00″		116° 19′ 12″
E	92° 04′30″ →	92° 05′00″	30″	92° 04′ 42″
	538° 119′00″	538° 120′00″		538° 118′120″
	= 539° 59′00″	= 540° 00′00″		= 540° 00′ 00″
	Error = 01′	Balanced		Balanced

$$\text{Correction/angle} = \frac{60}{5} = 12''$$

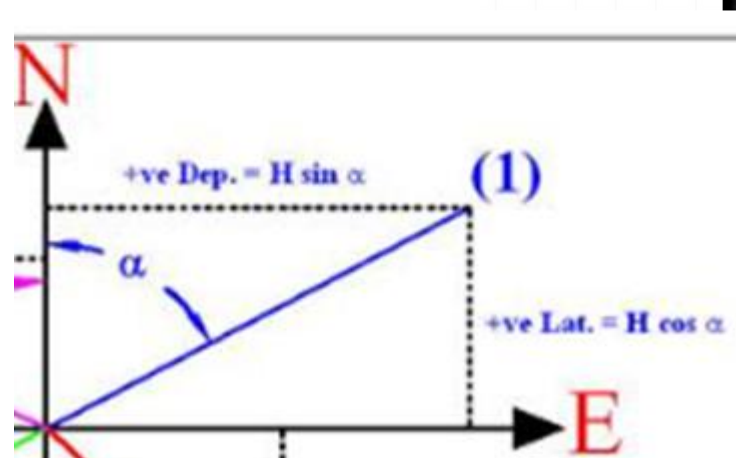
Latitudes & Departures

⇒ **Latitude:** North/South rectangular component of a line

N = +ve
S = –ve

⇒ **Departure:** East/West rectangular component

E = +ve
W = –ve



⇒ **Latitude (Δy) = dist (H) cos α**

⇒ **Departure (Δx) = dist (H) sin α**

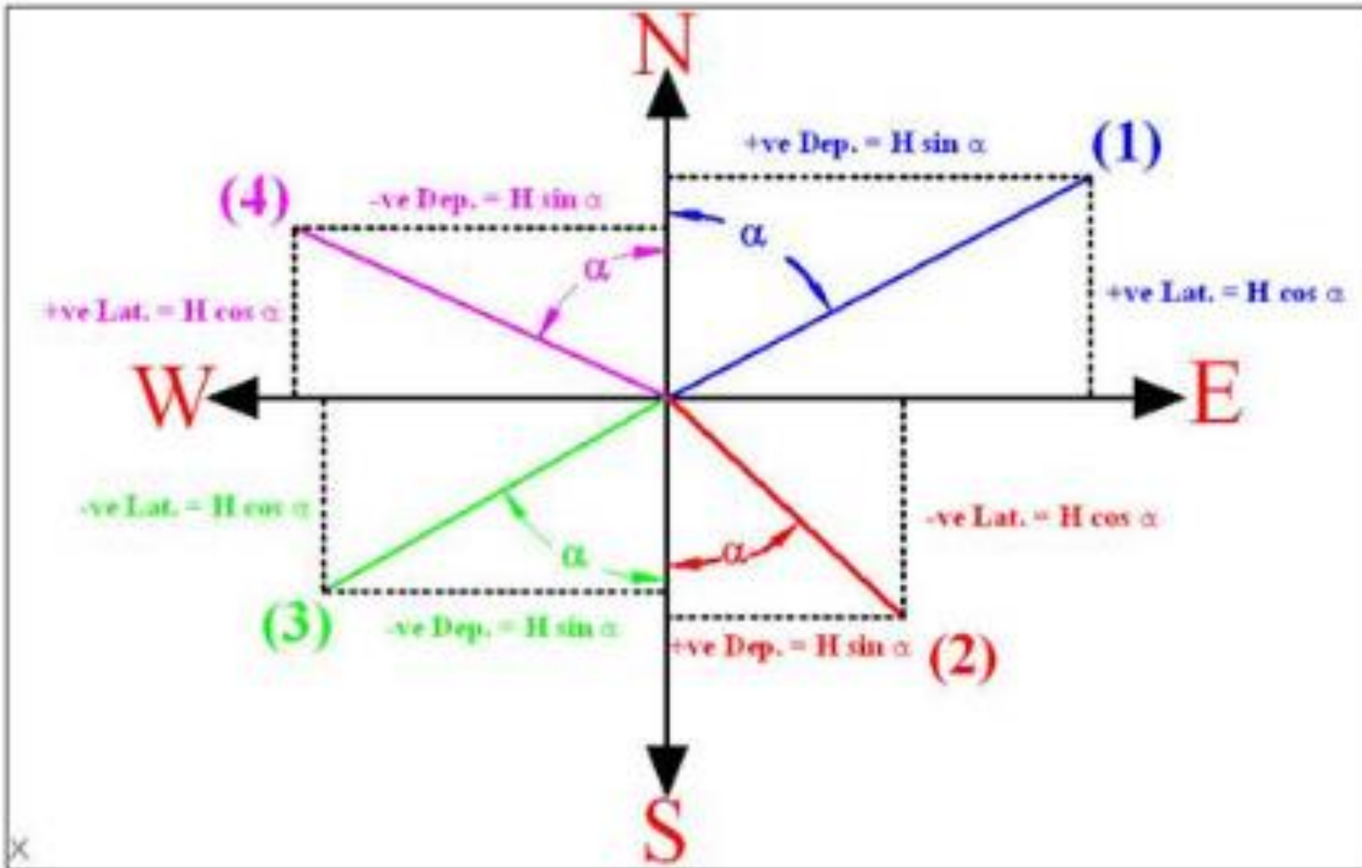
where, H ≡ horizontal dist of Traverse course

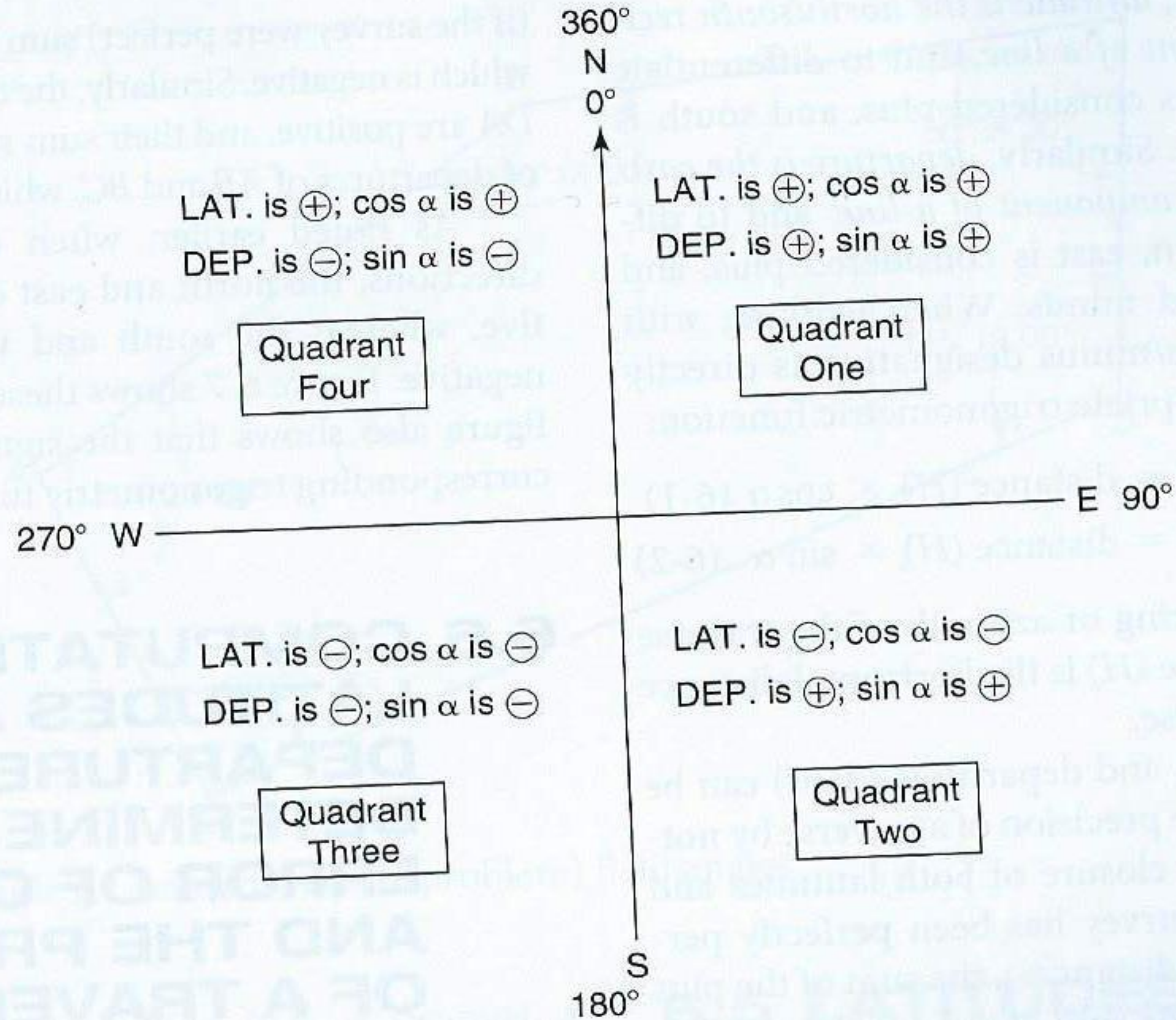
α ≡ is Azimuth (sign is automatically corrected)

or

Bearing (sign must be entered).

Latitudes & Departures



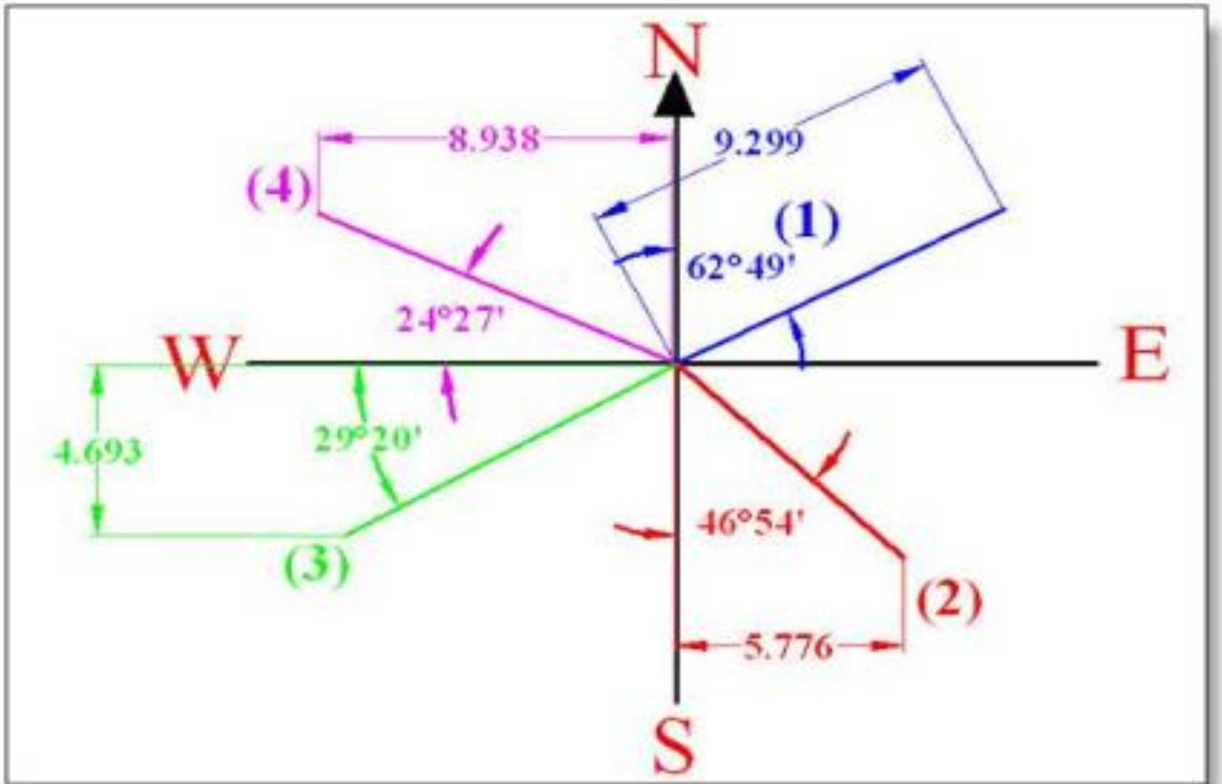


Algebraic signs of latitudes and departures by trigonometric functions (α is the azimuth)

Latitudes & Departures

Example

Q1. In a tabular form calculate the latitudes and departures of the following lines.



Latitudes & Departures Example: complete blank cells

line	length	Azimuth	Bearing	Latitude	Departure
1	9.299	62º 49'	N 62º 49'E		
2			S 46º 54' E		5.776
3				-4.693	
4					-8.938

Latitudes & Departures Example: Solution

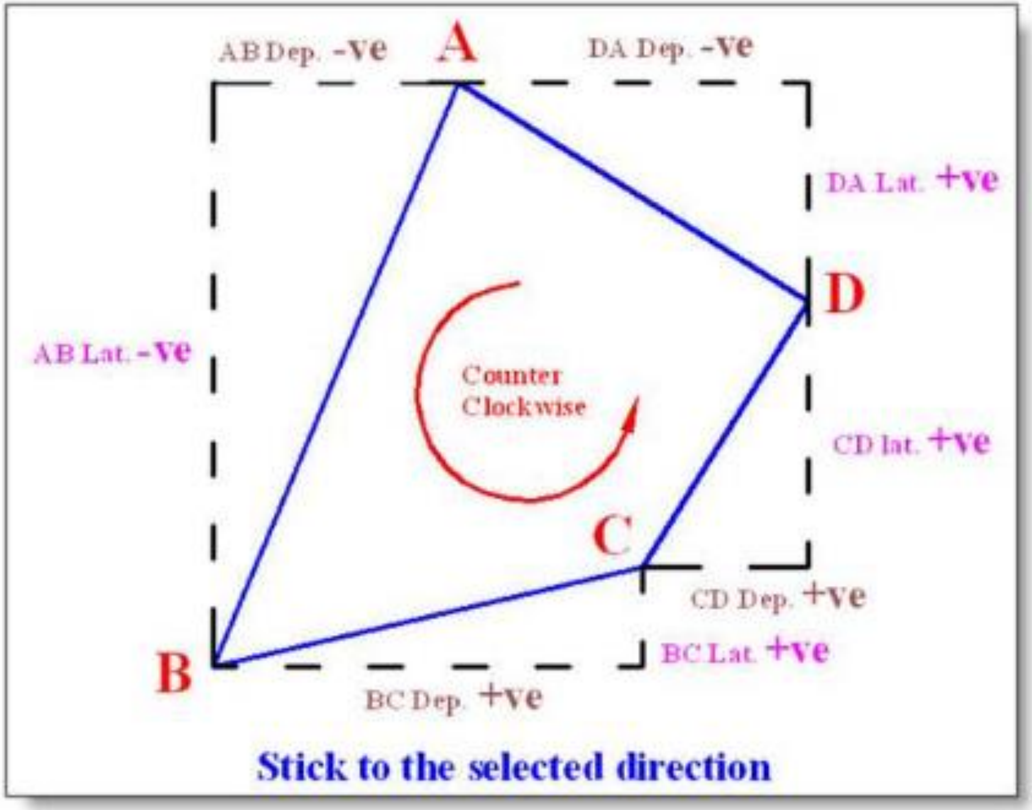
line	length	Azimuth	Bearing	Latitude H cos	Departure H sin
1	9.299	62° 49’	N 62° 49’E	4.248	8.272
2	7.911	133° 06’	S 46° 54’ E	-5.405	5.776
3	9.580	240° 40’	S 60° 40’ W	-4.693	-8.351
4	9.818	294° 27’	N 65° 33’ W	4.064	-8.938

Latitudes & Departures

⇒ In a perfect (or corrected) survey work;

$\sum$ +ve Latitudes = $\sum$ -ve Latitudes

$\sum$ +ve Departures = $\sum$ -ve Departures



The error of closure (linear error of closure): is the net accumulation of the random errors associated with the measurement of traverse angles and traverse distances.

The error of closure is compared to the perimeter of the traverse to determine the **precision ratio**

The fraction **E/P** is always expressed so that the numerator is 1, and the denominator is rounded to the closest 100 units.

Usually **E/P: 1/3000-1/10000** for engineering surveys

<u>If E/P is not within the permissible limits</u>:

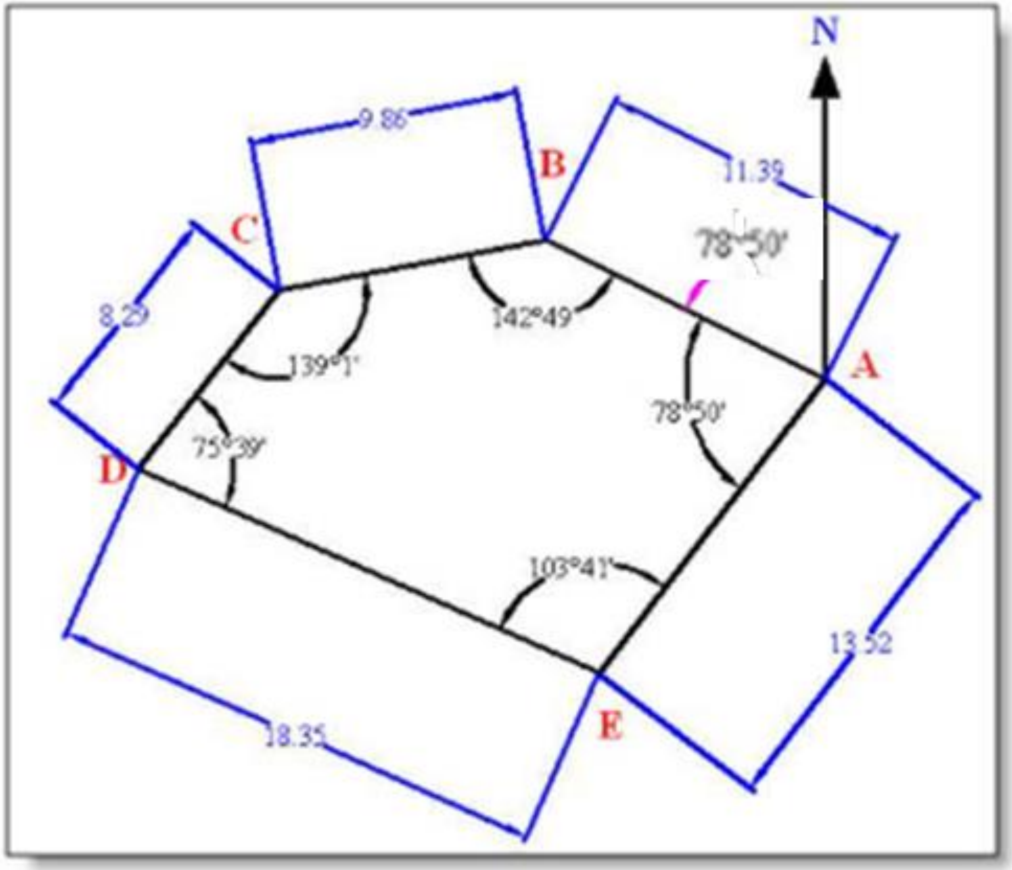
- Double-check all computations
- Double-check all field entries
- Compute the bearing of the linear error of closure and check to see if it is similar to a course bearing (±5°)
- Re-measure the sides of the traverse, beginning with a course having a bearing similar to the linear error of closure bearing (if there is one)
- When a correction is found for a measured side, try that value in the latitude-departure computation to determine the new level of precision.

Error of Closure

⇒ To find Linear Error of Closure (LEC) and Accuracy of a Traverse:

1. Balance the angles

2. Find azimuth (or bearing) of all traverse sides.



Error of Closure

Points	Angle	Azimuth	Bearing
A	78° 50'		
		281° 10'	N 78° 50' W
B	142° 49'		
		243° 59'	S 63° 59' W
C	139° 01'		
		203° 00'	S 23° 00' W
D	75° 39'		
		98° 39'	S 81° 21' E
E	103° 41'		
		22° 20'	N 22° 20' E
A	78° 50'		

Error of Closure

3. Find latitudes (Δy) and departures (Δx)

Point	Angle	Length (m)	Azimuth	Bearing	Latitude (Δy) = H cos α	Departure (Δy) = H sin α
A	78° 50’					
		11.39	281° 10’	N 78° 50’ W	2.21	-11.17
B	142° 49’					
		9.86	243° 59’	S 63° 59’ W	-4.32	-8.86
C	139° 01’					
		8.29	203° 00’	S 23° 00’ W	-7.63	-3.24
D	75° 39’					
		18.35	98° 39’	S 81° 21’ E	-2.76	18.14
E	103° 41’					
		13.52	22° 20’	N 22° 20’ E	12.52	5.01
A	78° 50’					
Σ	540° 00’	61.41			0.02	-0.12

Note that in the above table, you have to end up with the same angle you have started with. We have started and ended the table with point A.

Linear Error of Closure = LEC = $\sqrt{\sum lat^2 + \sum dep^2}$

$$= \sqrt{(0.02)^2 + (-0.12)^2}$$

$$= 0.12$$

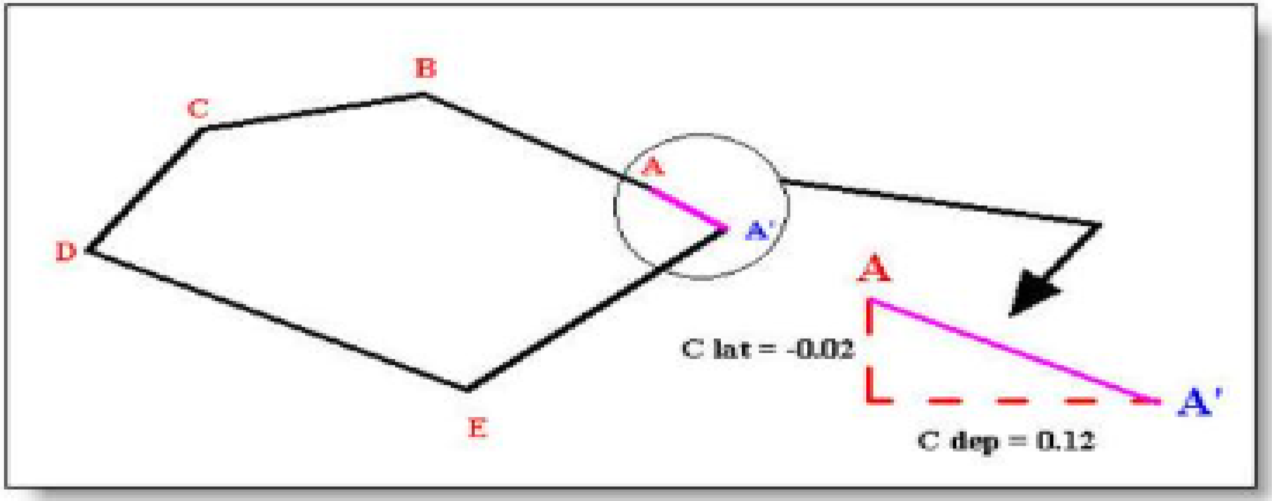
Accuracy Ratio = LEC / ∑H = 0.12 / 61.41
= 1 / 511.75 = 1 / 500

The total error in the latitudes = 0.02
The total error in the departures = -0.12
Therefore, you have to subtract these errors from the latitudes and departures (according to there weights) to have the corrected latitudes and departures.

Correction in Latitudes = C_{lat} = - Sum of latitudes = - 0.02
Correction in departures = C_{dep} = - Sum of departures = 0.12

If we draw the measured traverse to scale on a paper by starting from point A going counter clockwise to B, C, D, E then back to A, we find that starting A doesn't match ending A. This is due to the error of closure.

Error of Closure



$$AA' = \sqrt{C_{lat}^{\,2} + C_{dep}^{\,2}}$$

$$= \sqrt{(0.02)^2 + (-0.12)^2} = 0.12$$

$$bearing\ of\ AA' = \tan^{-1}\frac{C_{dep}}{C_{lat}}$$

$$= \tan^{-1}(0.12\ /\ -0.02)$$

$$= S\ 80^\circ\ 32'\ 16''E$$

Accuracy

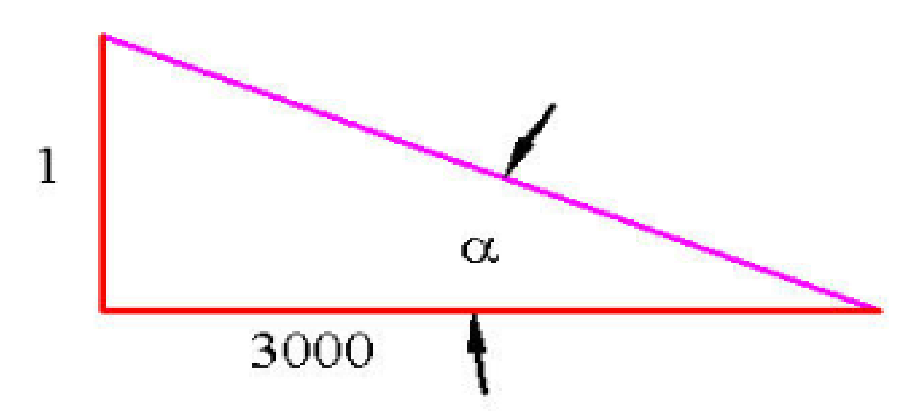
- Accuracy ratio not enough for ensuring accuracy!
- Errors might cancel each other
- So in addition to accuracy ratio:
  - Check max allowable error in angle E_a
  - Check overall max allowable angular error

$$=E_a \sqrt{n}$$

Example

For 5 sided traverse and linear accuracy 1/3000, find the allowable error in angles and overall max allowable angular error.

$$Ea = \alpha = \tan^{-1}\left(\frac{1}{3000}\right)$$



$$E_a = \alpha = \tan^{-1}(1/3000)$$

$$E_a = 0.0191^{\circ} = 1.1' = 1'$$

$$\text{Overall allowable error} = E_a\sqrt{n} = 1'\sqrt{5} = \underline{2}'$$

$\therefore$ **Error in any angle** $E_a \leq 1'$

and

Total error in all angles $\leq 2'$

Table 6-3 Linear and Angular Error Relationships

Linear Accuracy Ratio	Maximum Angular Error, E_a	Least Count of Total Station or Theodolite Scale or Readout
1/1,000	0°03′26″	01′
1/3,000	0°01′09″	01′
1/5,000	0°00′41″	30″
1/7,500	0°00′28″	20″
1/10,000	0°00′21″	20″
1/20,000	0°00′10″	10″

Traverse Adjustments

- Compass rule adjustment for lat & dep:-

$$\Rightarrow \text{Correction in Lat} = -(\text{error in Lat}) * \frac{H}{P}$$

$$\Rightarrow \text{Correction in Dep} = -(\text{error in Dep}) * \frac{H}{P}$$

where,

H $\Rightarrow$ side length (Horizontal distance)

P $\Rightarrow$ perimeter length ($\Sigma\ell$)

- **Once the latitudes and departures have been adjusted, the original polar coordinates (distance and direction) will no longer be valid**

- **The distances and directions should be corrected**

$$distance\ corrected = \sqrt{lat^2 + dep^2}$$

$$\tan(bearingangle) = \frac{dep}{lat}$$

Latitudes and departures correction of the previous example

Point	Angle	Length (m)	Azimuth	Bearing	Latitude (Δy) = H cos α	Departure (Δx) = H sin α	C_{lat}	C_{dep}	Balanced Latitude	Balanced Departure
A	78° 50'									
		11.39	281° 10'	N 78° 50' W	2.21	-11.17	- 0.02* (11.39/61.41) = - 0.004	+ 0.12* (11.39/61.41) = + 0.022	2.21	-11.15
B	142° 49'									
		9.86	243° 59'	S 63° 59' W	-4.32	-8.86	- 0.02* (9.86/61.41) = - 0.003	+ 0.12* (9.86/61.41) = + 0.019	-4.32	-8.84
C	139° 01'									
		8.29	203° 00'	S 23° 00' W	-7.63	-3.24	= - 0.003	= + 0.016	-7.63	-3.22
D	75° 39'									
		18.35	98° 39'	S 81° 21' E	-2.76	18.14	= - 0.006	= + 0.036	-2.77	18.18
E	103° 41'									
		13.52	22° 20'	N 22° 20' E	12.52	5.01	= - 0.004	= + 0.026	12.52	5.04
A	78° 50'									
Σ	540° 00'	61.41			0.02	-0.12			0.00	0.00

-

Traverse Adjustments

Correction of Original Traverse Data

$$\text{AB (corrected)} = \sqrt{(lat\,AB)^2 + (dep\,AB)^2}$$

$$= \sqrt{(2.21)^2 + (-11.15)^2} = \mathbf{11.37}$$

$$bearing\ of\ AB\,(corrected) = \tan^{-1}\frac{dep\ AB}{lat\ AB}$$

$$= \tan^{-1}\frac{-11.25}{2.21} = -78.886^{\circ}$$

$$= \text{N } 78^{\circ}\ 54'\ \text{W}$$

Point	Angle	Length (m)	Azimuth	Bearing	Latitude (Δy) = H cos α	Departure (Δx) = H sin α	Balanced Latitude	Balanced Departure	Corrected Distance	Corrected Bearing
A	78° 50’									
		11.39	281° 10’	N 78° 50’ W	2.21	-11.17	2.21	-11.15	11.37	N 78° 54’ W
B	142° 49’									
		9.86	243° 59’	S 63° 59’ W	-4.32	-8.86	-4.32	-8.84	9.84	S 63° 57’ W
C	139° 01’									
		8.29	203° 00’	S 23° 00’ W	-7.63	-3.24	-7.63	-3.22	8.28	S 22° 53’ W
D	75° 39’									
		18.35	98° 39’	S 81° 21’ E	-2.76	18.14	-2.77	18.18	18.39	S 81° 20’ E
E	103° 41’									
		13.52	22° 20’	N 22° 20’ E	12.52	5.01	12.52	5.04	13.50	N 22° 56’ E
A	78° 50’									
Σ	540° 00’	61.41			0.02	-0.12	0.00	0.00		

Coordinate Computation

- Rectangular coordinates define the position of a point with respect to two perpendicular axes.
- Coordinates using analytical geometry can be used for further computations.
- In surveys where a coordinate grid system is not available, assume the X & Y axes in a position to have positive coordinates of all stations.
- Start the coordinates computation by assuming a large value for the starting station, (i.e., 1000.00, 1000.00).
- In your tabulation, start and end with the starting station.
- Check that you have got the calculated coordinates of the starting station equal to the assumed ones.

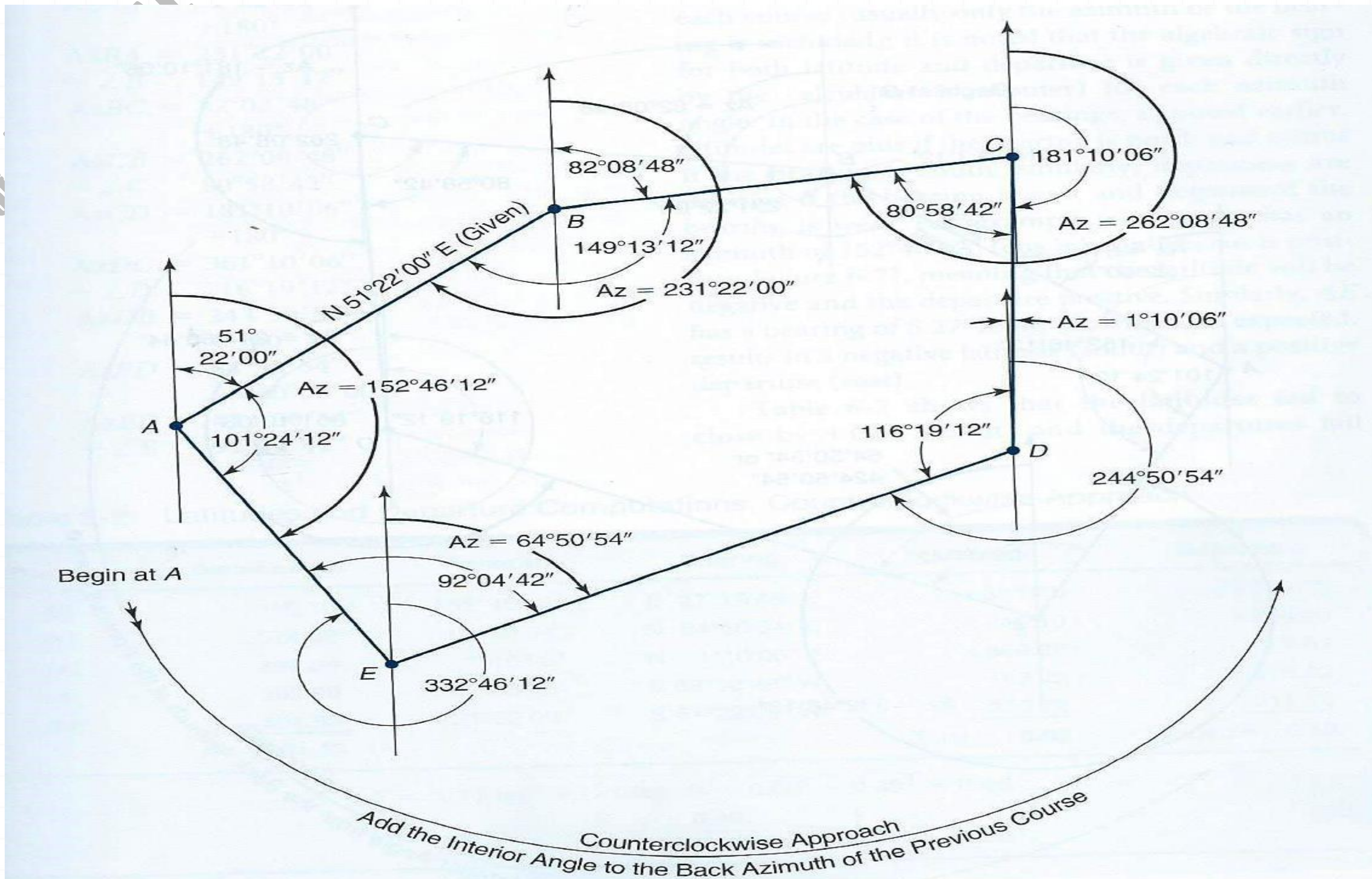
Latitudes and departures correction of the previous example

Point	Angle	Length (m)	Azimuth	Bearing	Lat.	Dep.	Corrected Latitude	Corrected Departure	Northing (y)	Easting (x)
A	78° 50'								1000.00	1000.00
		11.39	281° 10'	N 78° 50' W	2.21	-11.17	2.21	-11.15		
B	142° 49'								1000.00+2.21 = 1002.21	1000-11.15 = 988.85
		9.86	243° 59'	S 63° 59' W	-4.32	-8.86	-4.32	-8.84		
C	139° 01'								1002.21-4.32 = 997.89	988.85–8.84 = 980.01
		8.29	203° 00'	S 23° 00' W	-7.63	-3.24	-7.63	-3.22		
D	75° 39'								990.26	976.79
		18.35	98° 39'	S 81° 21' E	-2.76	18.14	-2.77	18.18		
E	103° 41'								987.49	994.97
		13.52	22° 20'	N 22° 20' E	12.52	5.01	12.52	5.04		
A	78° 50'								1000.00	1000.00
Σ	540° 00'	61.41			0.02	-0.12	0.00	0.00		

-

Coordinate calculation is OK; since the assumed coordinate of point A is equal to its calculated coordinate.

Example 2



N 51°22′00″ E (Given)
82°08′48″
149°13′12″
Az = 231°22′00″
181°10′06″
80°58′42″
Az = 262°08′48″
Az = 1°10′06″
116°19′12″
244°50′54″
51°
22′00″
Az = 152°46′12″
101°24′12″
Az = 64°50′54″
92°04′42″
332°46′12″
Begin at A
A
B
C
D
E
Counterclockwise Approach
Add the Interior Angle to the Back Azimuth of the Previous Course

Example 2

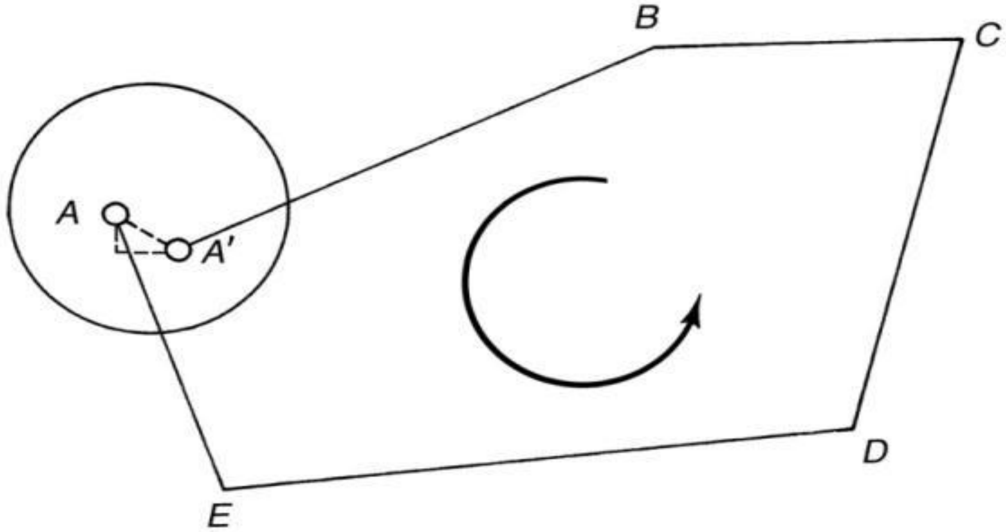
Table 6.2 LATITUDES AND DEPARTURE COMPUTATIONS, COUNTERCLOCKWISE APPROACH

Course	Distance (ft)	Azimuth	Bearing	Latitude	Departure
AE	350.10	152°46′12″	S 27°13′48″E	−311.30	+160.19
ED	579.03	64°50′54″	N 64°50′54″E	+246.10	+524.13
DC	368.28	1°10′06″	N 1°10′06″E	+368.20	+7.51
CB	382.20	262°08′48″	S 82°08′48″W	−52.22	−378.62
BA	401.58	231°22′00″	S 51°22′00″W	−250.72	−313.70
	P = 2081.19			Σ lat = +0.06	Σ dep = −0.49

$$E = \sqrt{\Sigma\, \text{lat}^2 + \text{dep}^2} = \sqrt{0.06^2 + 0.49^2} = 0.49$$

$$\text{Precision ratio} = \frac{E}{P} = \frac{0.49}{2081.19} = \frac{1}{4247} \approx \frac{1}{4200}$$

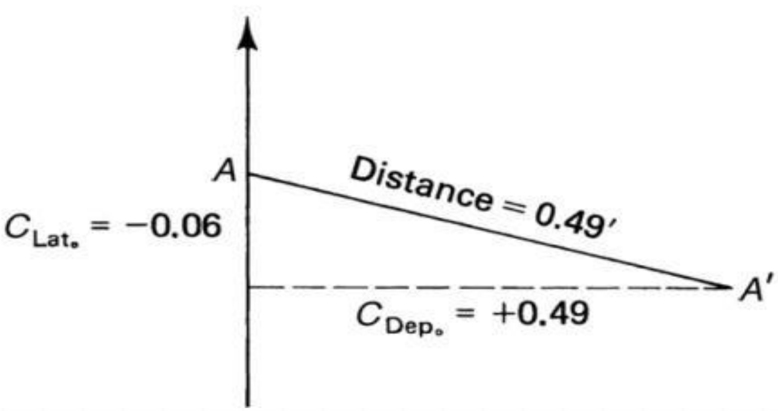
Example 2



Closure Error = $A'A$

Closure Correction = AA'

Solution Proceeds Counterclockwise Around the Traverse beginning at A.



$$AA' = \sqrt{C_{Lat.}{}^2 + C_{Dep.}{}^2} = 0.494'$$

Bearing of AA' Can Be Computed from the Relationship:

$$\tan \text{Bearing} = \frac{C_{Dep.}}{C_{Lat.}} = \frac{0.49}{-0.06}$$

Bearing Angle = 83.0189° = 83°01′

Bearing AA' = S 83°01′ W

Example 2

Table 6-4 Traverse Adjustments: Compass Rule, Section 6.6

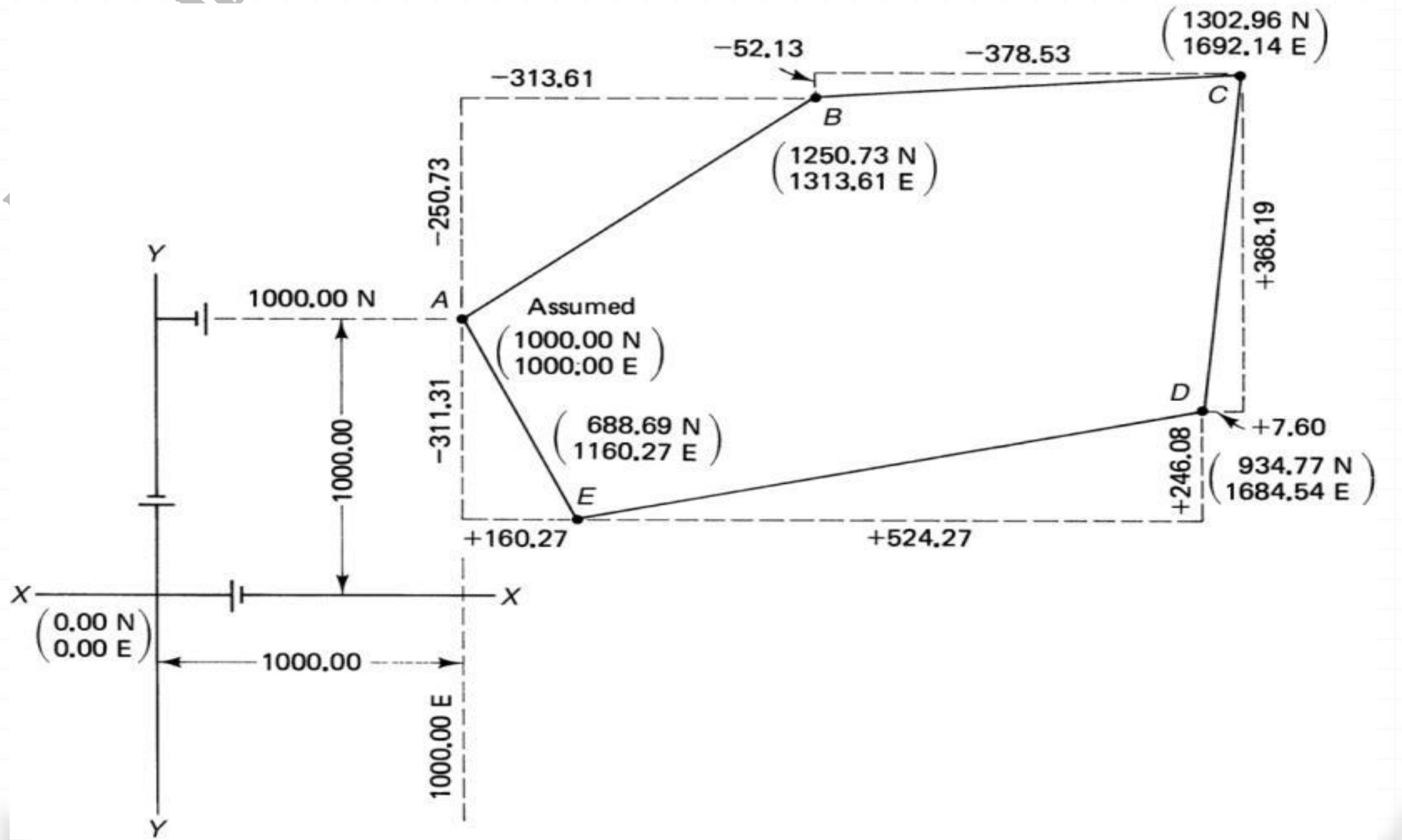
Course	Distance (ft)	Bearing	Latitude	Departure	C lat	C dep	Balanced Latitudes	Balanced Departures
AE	350.10	S 27°13′48″ E	−311.30	+160.19	−0.01	+0.08	−311.31	+160.27
ED	579.03	N 64°50′54″ E	+246.10	+524.13	−0.02	+0.14	+246.08	+524.27
DC	368.28	N 1°10′06″ E	+368.20	+7.51	−0.01	+0.09	+368.19	+7.60
CB	382.20	S 82°08′48″ W	−52.22	−378.62	−0.01	+0.09	−52.23	−378.53
BA	401.58	S 51°22′00″ W	−250.72	−313.70	−0.01	+0.09	−250.73	−313.61
	$P = 2081.19$		$\sum lat = +0.06$	$\sum dep = -0.49$	$\sum C_{lat} = -0.06$	$\sum C_{dep} = +0.49$	0.00	0.00

Example 2

Table 6-5 Adjustment of Bearings and Distances Using Balanced Latitudes and Departures: (Section 6.6)

Course	Balanced Latitude	Balanced Departure	Adjusted Distance (ft)	Adjusted Bearing	Original Distance (ft)	Original Bearing
AE	−311.31	+160.27	350.14	S 27°14′26″ E	350.10	S 27°13′48″ E
ED	+246.08	+524.27	579.15	N 64°51′21″ E	579.03	N 64°50′54″ E
DC	+368.19	+7.60	368.27	N 1°10′57″ E	368.28	N 1°10′06″ E
CB	−52.23	−378.53	382.12	S 82°08′38″ W	382.20	S 82°08′48″ W
BA	−250.73	−313.61	401.52	S 51°21′28″ W	401.58	S 51°22′00″ W
	0.00	0.00	*P* = 2081.20		*P* = 2081.19	

Example 2


1302.96 N
1692.14 E
−52.13
−378.53
−313.61
C
B
1250.73 N
1313.61 E
−250.73
+368.19
Y
1000.00 N
A
Assumed
1000.00 N
1000.00 E
D
−311.31
1000.00
688.69 N
1160.27 E
+7.60
934.77 N
1684.54 E
+246.08
E
+160.27
+524.27
X
X
0.00 N
0.00 E
1000.00
1000.00 E
Y

Example 2

Table 6-8 Computation of Coordinates Using Balanced Latitudes and Departures

Course	Balanced Latitude	Balanced Departure	Station	North	East
			A	**1000.00** (assumed)	**1000.00** (assumed)
AE	−311.31	+160.27		−311.31	+160.27
			E	**688.69**	**1160.27**
ED	+246.08	+524.27		+246.08	+524.27
			D	**934.77**	**1684.54**
DC	+368.19	+7.60		+368.19	+7.60
			C	**1302.96**	**1692.14**
CB	−52.23	−378.53		−52.23	−378.53
			B	**1250.73**	**1313.61**
BA	−250.73	−313.61		−250.73	−313.61
			A	**1000.00 Check**	**1000.00 Check**

Missing (Omitted) Measurement

- Due to presence of obstacles like trees, water tunnel, or highway, sometimes it is difficult to directly measure one side of the traverse.

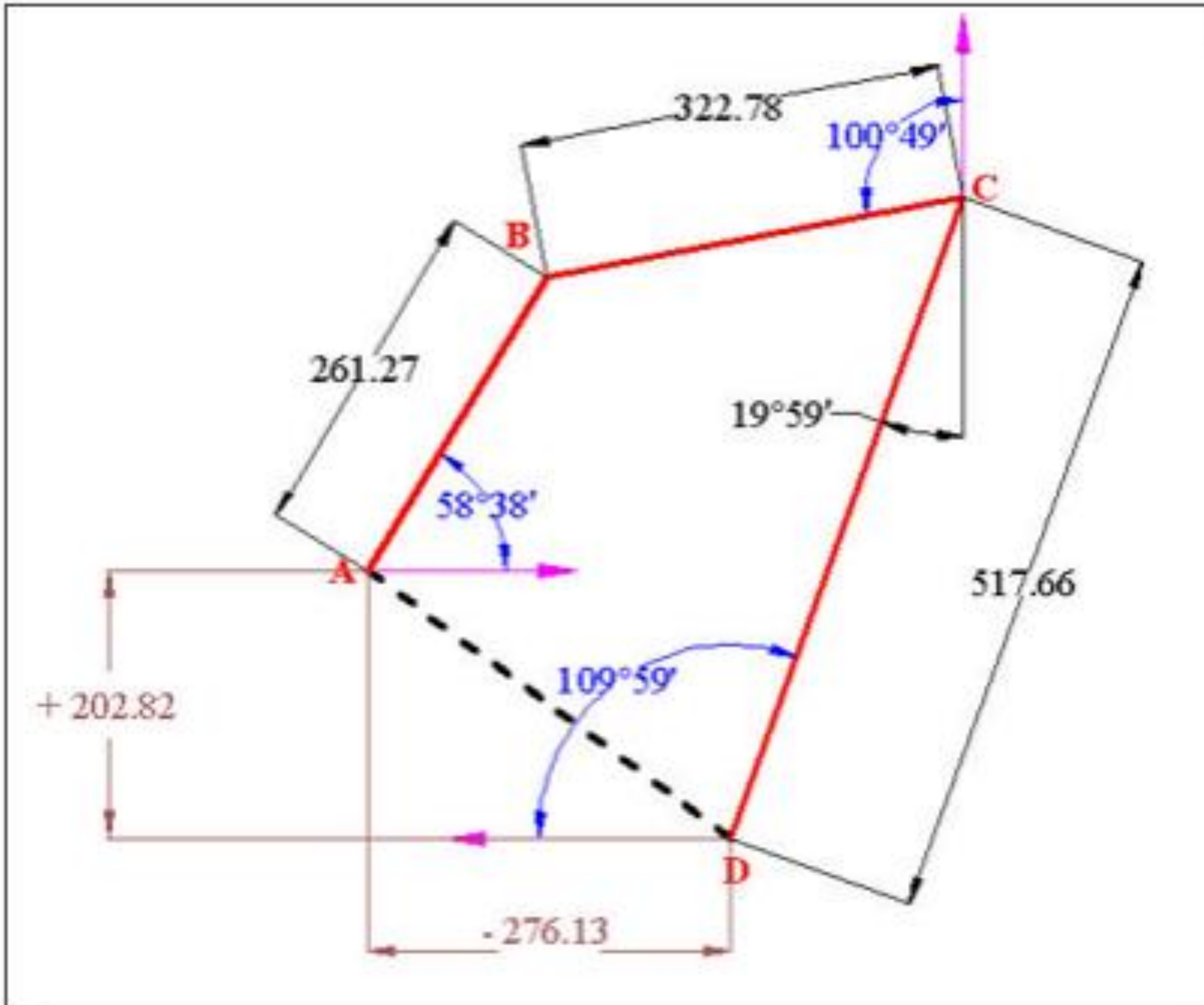
- The technique of latitudes and departures can be used to find that side, and complete the traverse.

- The idea is to arrange the sides in a form of closed traverse with one side missing.

- Mainly you have to stick to a certain direction (clockwise or counterclockwise) in naming and solving the traverse.

Missing Measurement

Example 1. Find the missing course in the following traverse:



Point	Length (m)	Bearing	Azimuth	Latitude (Δy) = H cos α	Departure (Δx) = H sin α
A					
	261.27	N 31° 22' E How?	31° 22'	+ 223.09	+ 135.99
B					
	322.78	N 79° 11' E	79° 11'	+ 60.58	+ 317.05
C					
	517.66	S 19° 59' W	199° 59'	- 486.49	- 176.91
D					
Σ				-202.82	+ 276.13
Corrected				+ 202.82	- 276.13
A	??	??		+ 202.82	- 276.13
Σ	61.41			0.02	-0.12

Note that we have stuck with the direction and that distances are placed in the cells between the points.

$$\text{DA} = \sqrt{DA_{lat}^{\ 2} + DA_{dep}^{\ 2}}$$

Note it is DA not AD. Why?

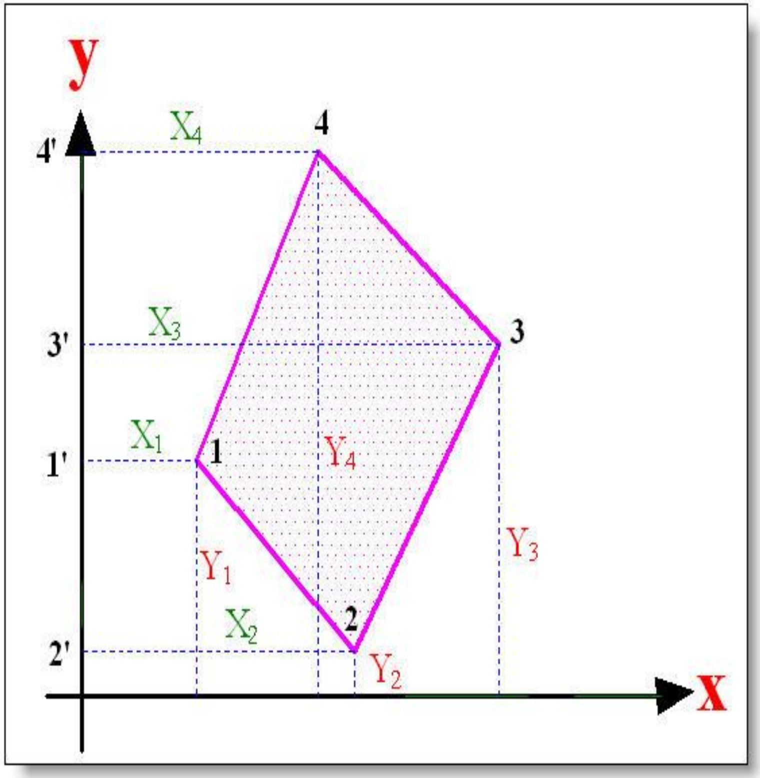
$$= \sqrt{(202.82)^2 + (-276.13)^2} = 342.61$$

$$bearing\ of\ DA = \tan^{-1}\frac{DA_{dep}}{DA_{lat}}$$

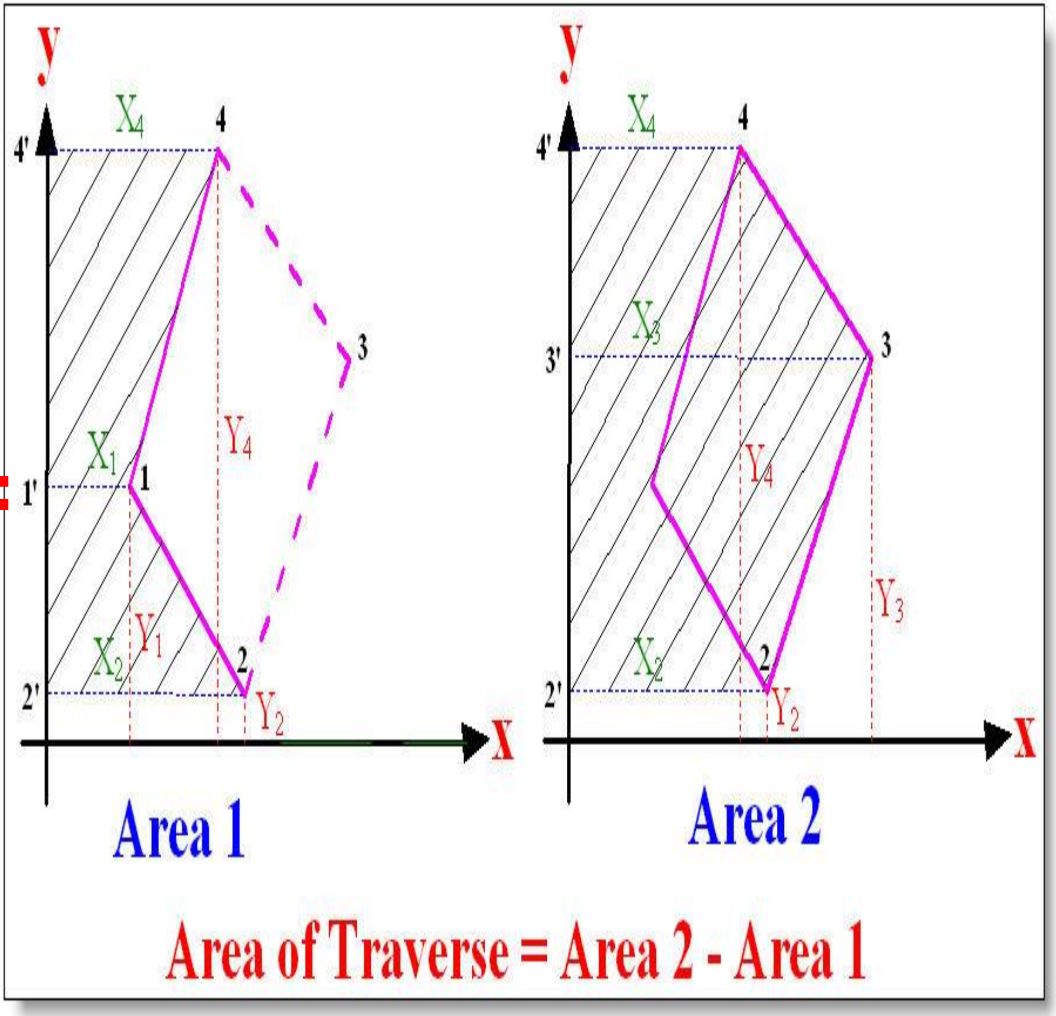
$$= \tan^{-1}\frac{-276.13}{+202.82}$$

$$= \text{N } 53° \ 42' \text{ W}$$

Area of a closed traverse by the coordinate method

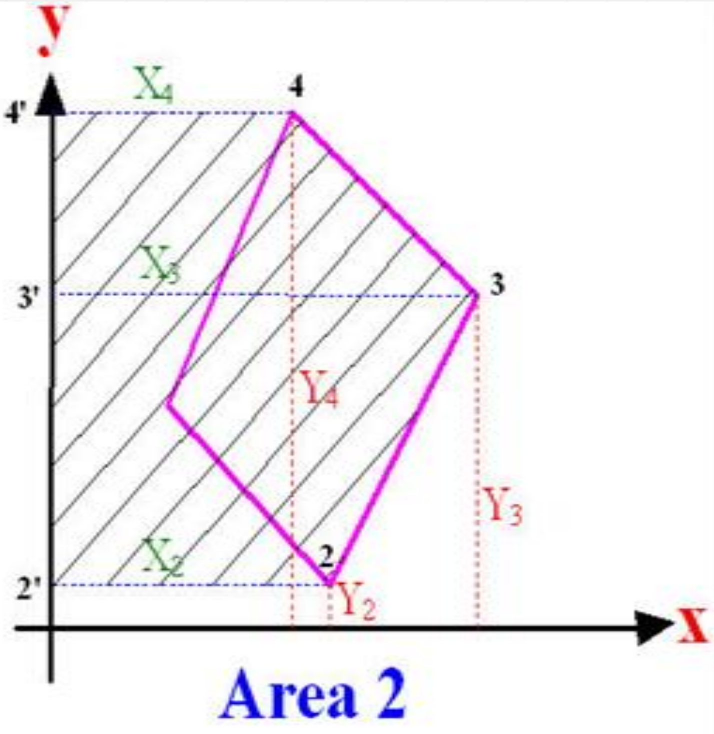


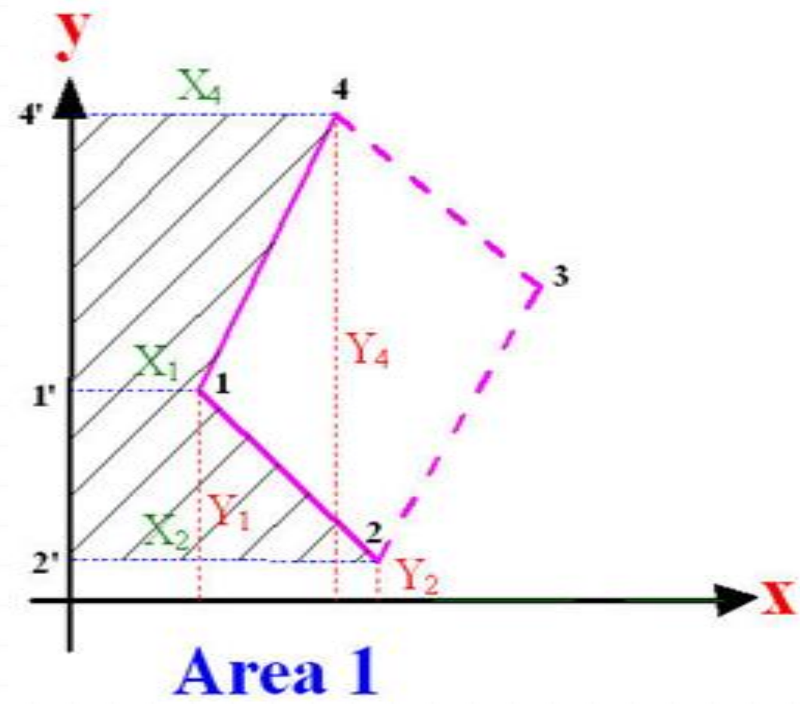
=



Area 2= area of trapezoid 4'433'+
area of trapezoid 3'322'

Area 1= area of trapezoid 4'411'+
area of trapezoid 1'122'





Area 2= area of trapezoid 4'433'+ area of trapezoid 3'322'

= ½ (X4+X3)(Y4-Y3) + ½ (X3+X2)(Y3-Y2)

Area 1= area of trapezoid 4'411'+ area of trapezoid 1'122'

= ½ (X4+X1)(Y4-Y1) + ½ (X1+X2)(Y1-Y2)

Area = [½ (X4+X3)(Y4-Y3) + ½ (X3+X2)(Y3-Y2)] - [½ (X4+X1)(Y4-Y1) + ½ (X1+X2)(Y1-Y2)]

Multiplying both sides by 2:

2 Area = [(X4+X3)(Y4-Y3) + (X3+X2)(Y3-Y2)] - [(X4+X1)(Y4-Y1) + (X1+X2)(Y1-Y2)]

Expanding the expression and collecting the remaining terms:

2 Area = [(X4+X3)(Y4-Y3) + (X3+X2)(Y3-Y2)] - [(X4+X1)(Y4-Y1) + (X1+X2)(Y1-Y2)]

2 Area= X4Y4-X4Y3+X3Y4-X3Y3+X3Y3-X3Y2+X2Y3-X2Y2

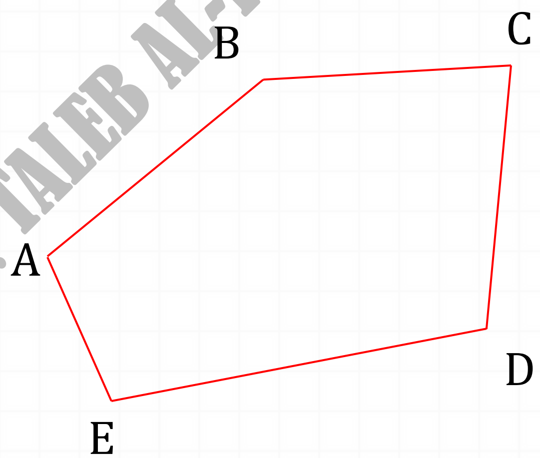
- [X4Y4-X4Y1+X1Y4-X1Y1+X1Y1-X1Y2+X2Y1-X2Y2]

→**2 Area = X1(Y2-Y4)+X2(Y3-Y1)+X3(Y4-Y2)+X4(Y1-Y3)**

Simply, the double area of a closed traverse is the algebraic sum of each x coordinate multiplied by the difference between the y coordinates of the adjacent stations.

The final area can result in a positive or a negative number, reflecting only the direction of computation (CW or CCW). The physical area is, of course, positive.

Example:



Calculate the area of the closed traverse ABCDEA Using the area by coordinates method?

XA(YB-YE)= 1000(1250.73-688.69)=+562040
XB(YC-YA)= 1313.61(1302.96-1000)=+397971
XC(YD-YB)= 1692.14(934.77-1250.73)=-534649
XD(YE-YC)= 1684.54(688.69-1302.96=-1034762
XE(YA-YD)= 1160.27(1000-934.77)=+75684

CW -533716 ft^2
2A= 533716 ft^2

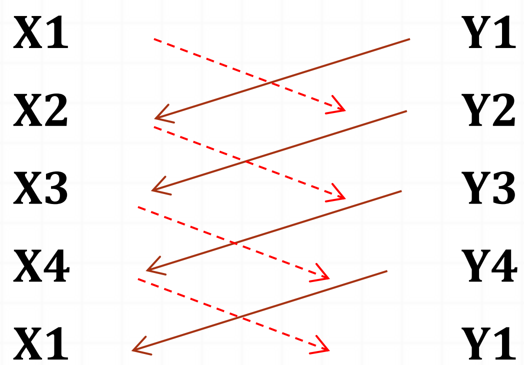
Area= 266858 ft^2

Station	North	East
A	1000.00 ft	1000.00 ft
B	1250.73	1313.61
C	1302.96	1692.14
D	934.77	1684.54
E	688.69	1160.27

Area of a closed traverse by the coordinate method

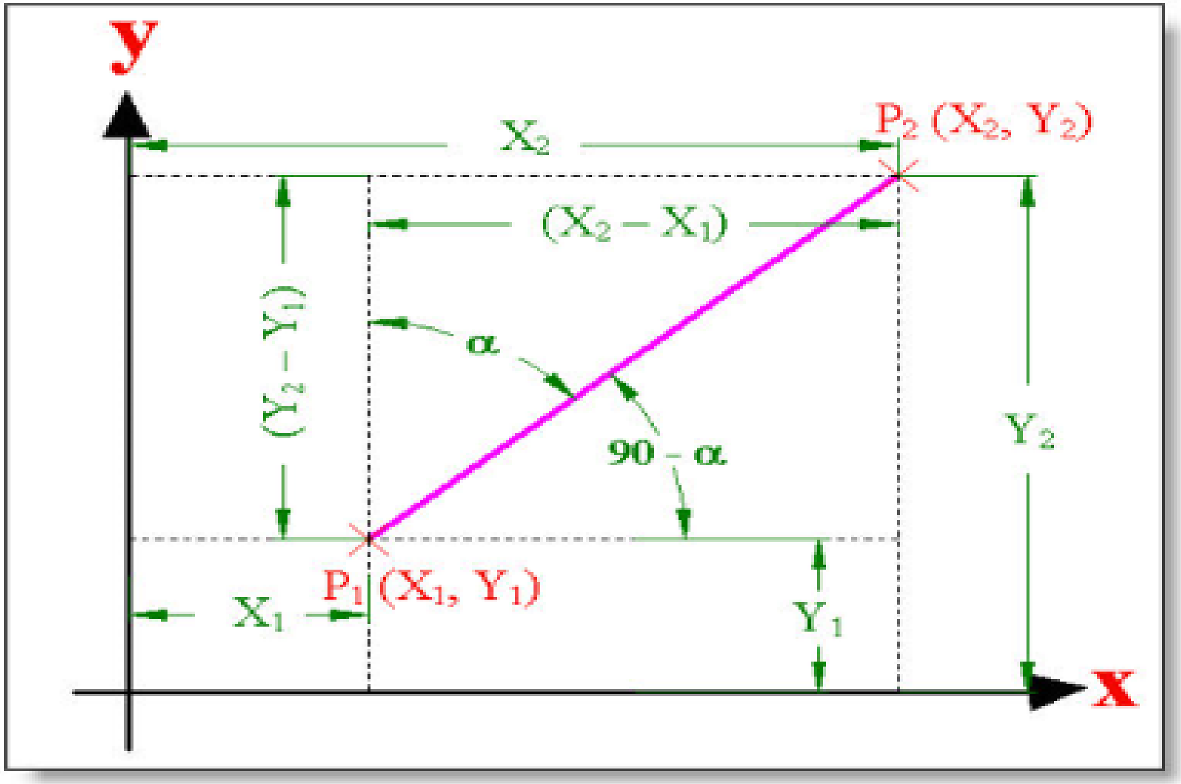
Equation can be reduced to an easily remembered form by listing the *X* and *Y* coordinates of each point in succession in two columns, *with coordinates of the starting point repeated at the end*.

The products noted by diagonal arrows are ascertained with dashed arrows considered plus and solid ones minus. *The algebraic summation of all products is computed and its absolute value divided by 2 to get the area*.



2 AREA = X1Y2 + X2Y3 + X3Y4 + X4Y1 - X2Y1 -X3Y2 - X4Y3 - X1Y4

Geometry of Rectangular Coordinates



⇒ **Equation of line P_1P_2:**

$$\frac{y - y_1}{x - x_1} = \frac{y_2 - y_1}{x_2 - x_1} \qquad (1)$$

⇒ **Length of line P_1P_2 =** $\sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2}$ **(2)**

⇒ $$\tan\alpha = \frac{x_2 - x_1}{y_2 - y_1} = \frac{dep}{lat} \qquad (3)$$

$\Rightarrow$ Slope of line $P_1P_2 = \dfrac{y_2 - y_1}{x_2 - x_1} = \text{Cot}\,\alpha$

$= 1/\tan\alpha \equiv m$ (4)

$\therefore$ From (1) & (4)

$$\frac{y - y_1}{x - x_1} = \frac{y_2 - y_1}{x_2 - x_1} = \text{Cot}\,\alpha \qquad (5)$$

$\Rightarrow$ $\therefore y - y_1 = \text{Cot}\,\alpha\,(x - x_1)$ (6)

Line $\perp$ to line of (6):

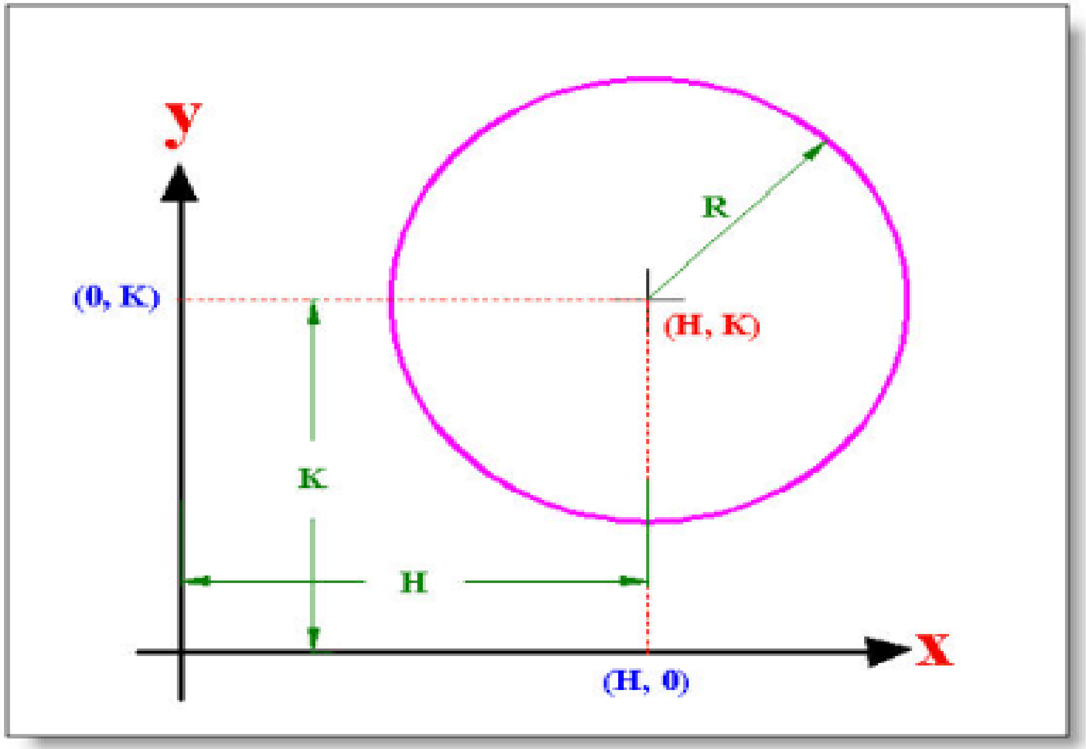
$$y - y_1 = -\tan\alpha\,(x - x_1) \qquad (7)$$

$\Rightarrow$ Equations (6) & (7) are same equation except the slope term (m) is (-1 / m)

$\Rightarrow$ dep. $= H \sin\alpha$ (8)

$\Rightarrow$ lat. $= H \cos\alpha$ (9)

Equation of Circular Curve:



$$(x - H)^2 + (y - K)^2 = r^2 \qquad (10)$$

where,

$r \equiv$ radius

H, K $\equiv$ coordinate of the center

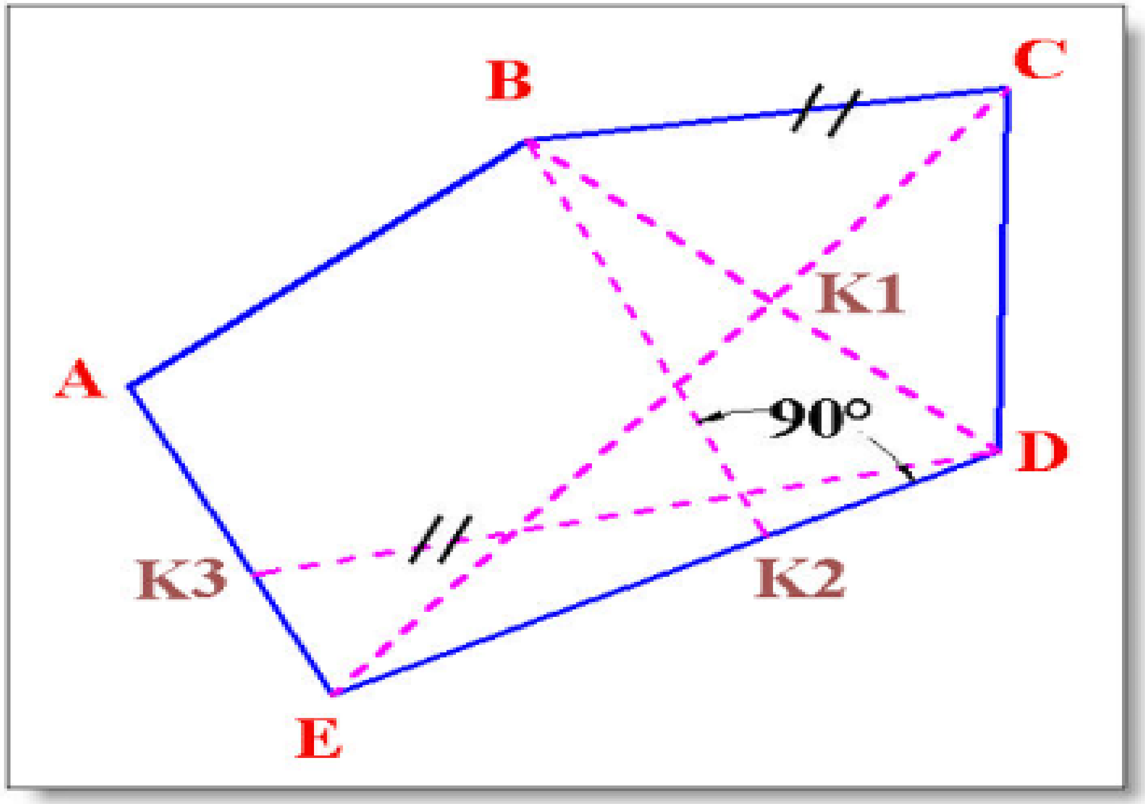
Equation of Circular Curve If center at origin $\Rightarrow$

$$x^2 + y^2 = r^2 \qquad (11)$$

Example 1.

For the following figure and stations' coordinates find the following:

- Find the coordinates of points K1, K2 & K3.
- Find K2D & K2E distances.



Station	Northen (y)	Easten (x)
A	1000.00	1000.00
B	1250.73	1313.61
C	1302.9	1692.14
D	934.77	1684.54
E	688.69	1160.27

Procedure for solution:

i) Coordinates of points K1, K2 & K3

$$\frac{y - y_1}{x - x_1} = \frac{y_2 - y_1}{x_2 - x_1}$$

From coordinates of points E & C ⇒ get equation of EC (1)

From coordinates of points D & B ⇒ get equation of DB (2)

Solve (1) & (2) ⇒ get point K1 coordinates.

From coordinates of points E & D ⇒ get equation of ED (3)

$$y - y_1 = m\,(x - x_1)$$

$$\therefore \quad \text{Equation of BK2} \Rightarrow \quad y - y_1 = -1/m\,(x - x_1) \qquad (4)$$

Solve (3) & (4) ⇒ get point K2 coordinates.

From coordinates of points C & B ⇒ get equation of CB

$$y - 1302.96 = (-52.23 / -378.53)\,(x - 1692.14)$$

(-52.23 / -378.53) = slope of BC = m = slope of DK3

$$\therefore \text{Equation of DK3} \Rightarrow y - D_y = m\,(x - D_x) \qquad (5)$$

From coordinates of points E & A ⇒ get equation of EA (6)

Solve (5) & (6) ⇒ get point K3 coordinates.

Q1. Numerically solve this example.

ii) K2D & K2E distances

Use coordinates of points K2, D & E

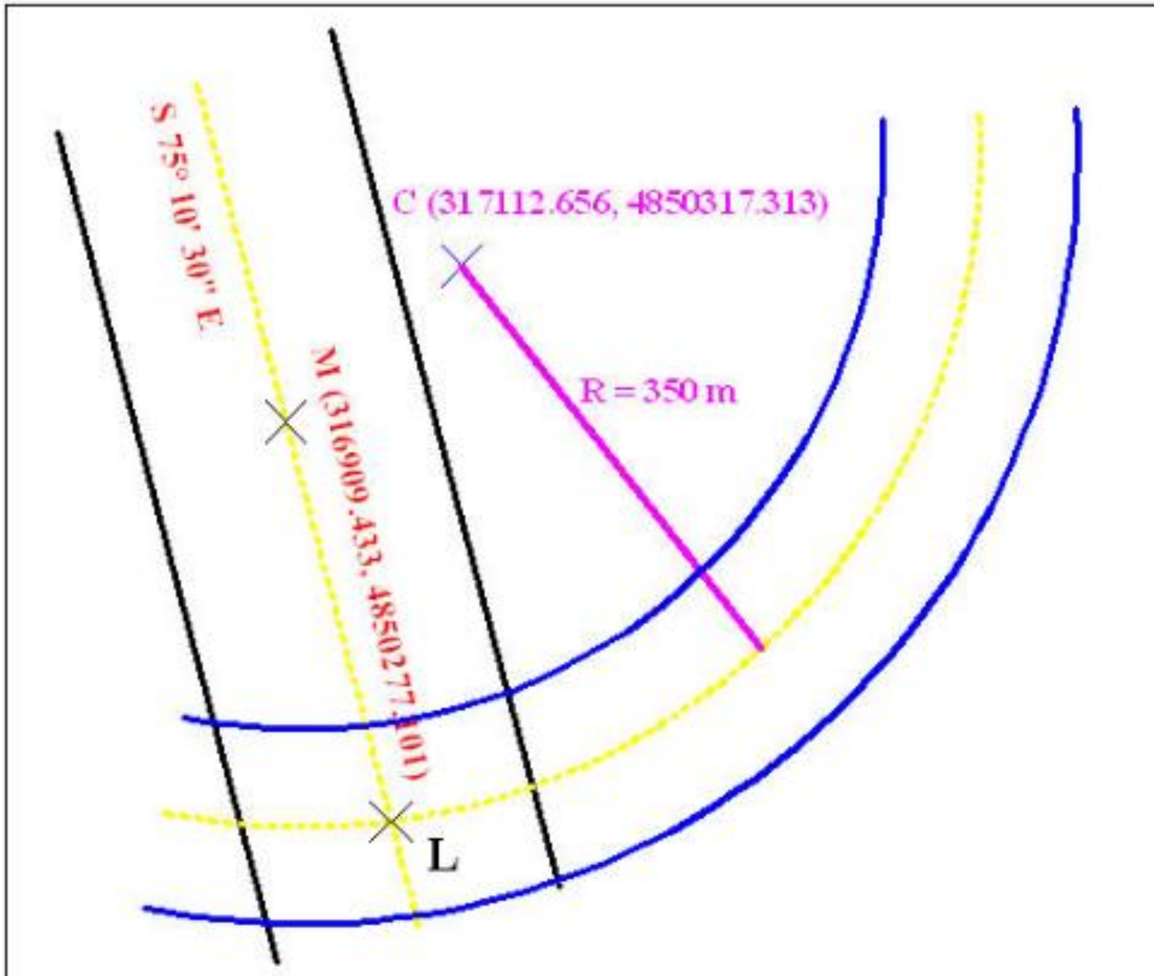
$$\text{and } l = \sqrt{(x_2 - x_1)^2 + (y_2 - y_1)^2}$$

get K2D & K2E distances.

- **Answers:**
- **k1 : (1087.33N, 1505.44E)**
- **k2 :(849.15 N, 1502.10 E)**
- **k3 :(850.91 N, 1076.75 E)**
- $length\ K2D = \sqrt{85.62^2 + 182.44^2} = 201.53$
- $length\ K2E = \sqrt{160.46^2 + 341.83^2}$
- **K2D+K2E=ED= 579.15**

Example 2.

From the information shown in the following figure, calculate the coordinates of the point of intersection (L) of the centerlines of the straight and circular road sections.



Example 2

- Procedure for solution:

- Since the coordinate values are very large which will cause significant rounding errors, it is recommended to refer them to a transferred local coordinate system. At the end of solution, the calculated coordinates will be referred back to the original coordinate system.
- The transferred coordinate system will be at (316500.00, 4850.00). Therefore the reduced coordinates are:

- M (409.433, 277.101)
- C (612.656, 317.313)

- From the slope of the straight section and point M coordinates Get the equation of the straight section. (1)

- From the radius of the circular section and center coordinates Get the equation of the circular section. (2)

- Solve (1) & (2) Þ get point L coordinates.

From the information shown in the following figure, calculate the coordinates of the point of intersection (L) of the centerlines of the straight and circular road sections.

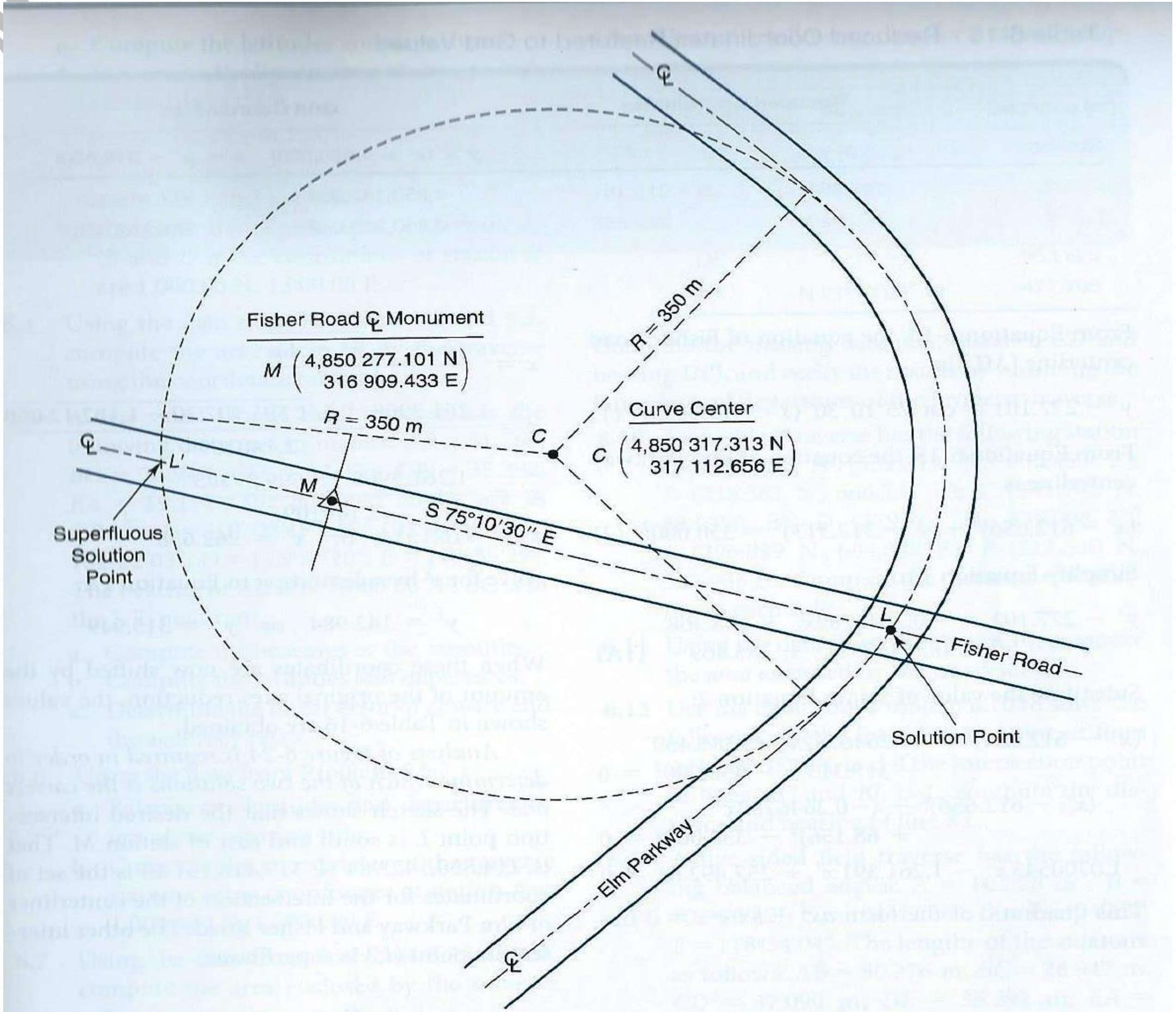


FIGURE 6-24 Intersection of a straight line with a circular curve, Example 6.9

Table 6-15 Grid Coordinates Reduced for Computations Using a Calculator

	Grid Coordinates		Reduced Coordinates	
Station	y	x	y' = (y − 4,850,000)	x' = (x − 316,500)
M	4,850,277.101	316,909.433	277.101	409.433
C	4,850,317.313	317,112.656	317.313	612.656

From Equation 6-13, the equation of Fisher Road centerline (*ML*) is

$$y' - 277.101 = \cot 75°10'30''(x' - 409.433) \quad (1)$$

From Equation 6-15, the equation of Elm Parkway centerline is

$$(x' - 612.656)^2 + (y' - 317.313)^2 = 350.000^2 \quad (2)$$

Simplify Equation 1 to

$$y' - 277.101 = -0.2646782x' + 108.368$$
$$y' = 0.2646782x' + 385.469 \quad (1A)$$

Substitute the value of y' into Equation 2:

$$(x' - 612.656)^2 = (0.2646782x' + 385.469 - 317.313)^2 - 350.000^2 = 0$$

$$(x' - 612.656)^2 + (-0.2646782x' + 68.156)^2 - 350.000^2 = 0$$

$$1.0700545\,x^2 - 1{,}261.391\,x' + 257{,}492.61 = 0$$

This quadratic of the form $ax^2 + bx + c = 0$ has roots

$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$x = \frac{1{,}261.3908 \pm \sqrt{1{,}591{,}107.30 - 1{,}102{,}124.50}}{2.140109}$$

$$x' = \frac{1{,}261.3908 \pm 699.27305}{2.140109}$$

$$x' = 916.1514 \quad \text{or} \quad x' = 262.658$$

Solve for y' by substituting in Equation 1A:

$$y' = 142.984 \quad \text{or} \quad y' = 315.949$$

When these coordinates are now shifted by the amount of the original axes reduction, the values shown in Table 6-16 are obtained.

Analysis of Figure 6-24 is required in order to determine which of the two solutions is the correct one. The sketch shows that the desired intersection point *L* is south and east of station *M*. That is, *L*(4,850,142.984 N, 317,416.151 E) is the set of coordinates for the intersection of the centerlines of Elm Parkway and Fisher Road. The other intersection point (*L'*) is superfluous.

Table 6-16 Reduced Coordinates Restored to Grid Values

Station	Reduced Coordinates		Grid Coordinates	
	y'	x'	$y = (y' + 4{,}850{,}000)$	$x = (x' + 316{,}500)$
L	142.984	916.151	4,850,142.984	317,416.151
L'	315.949	262.658	4,850,315.949	316,762.658

Prof. TALEB AL-ROUSAN

Surveying

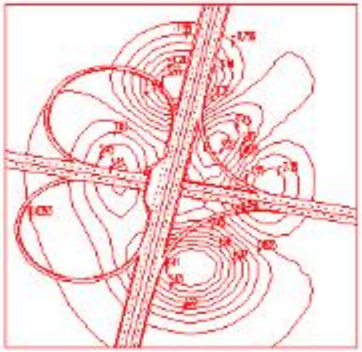
2104011365

Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University

Prof. TALEB AL-ROUSAN

Chapter 7
Topographic Survey

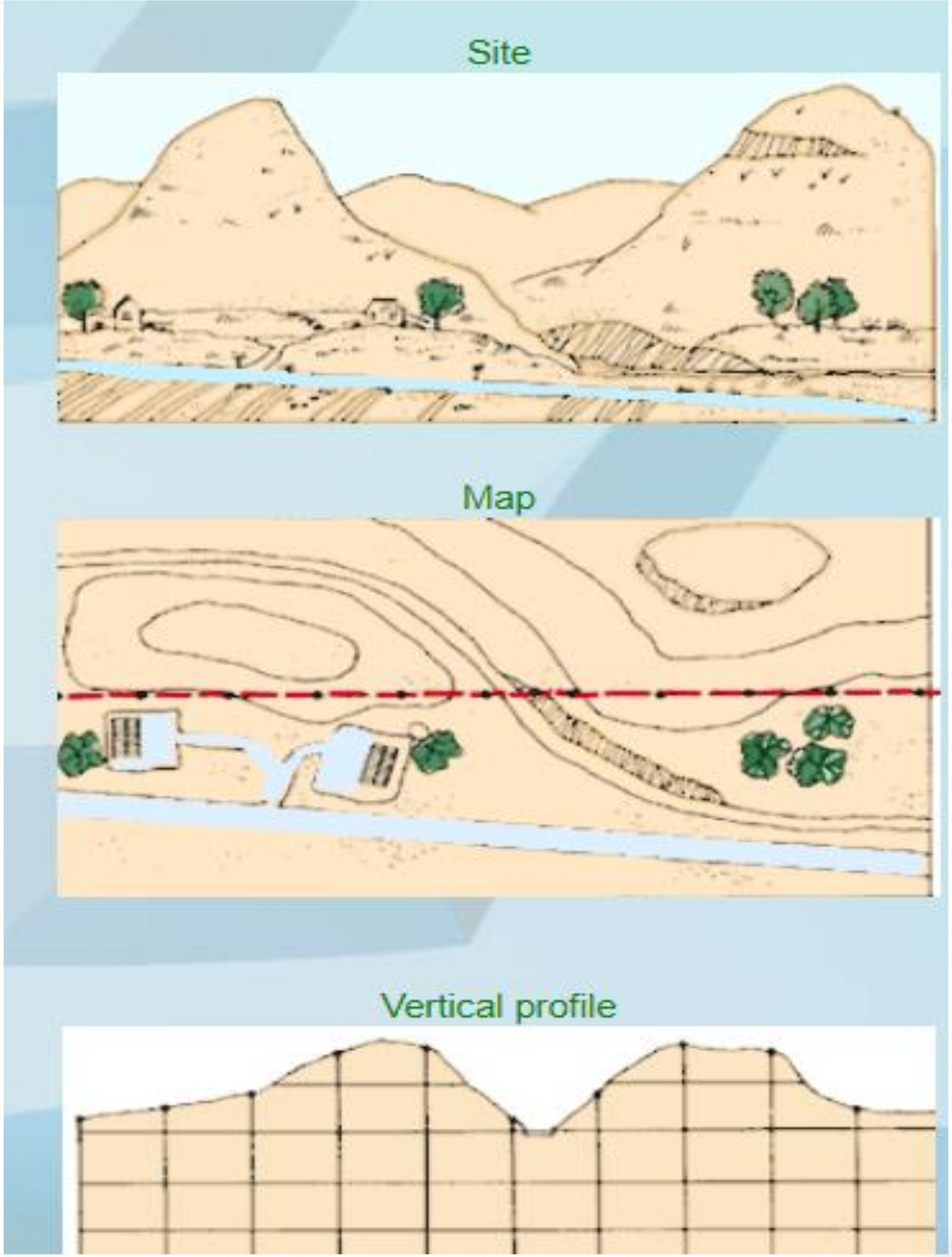
Topographic Surveys



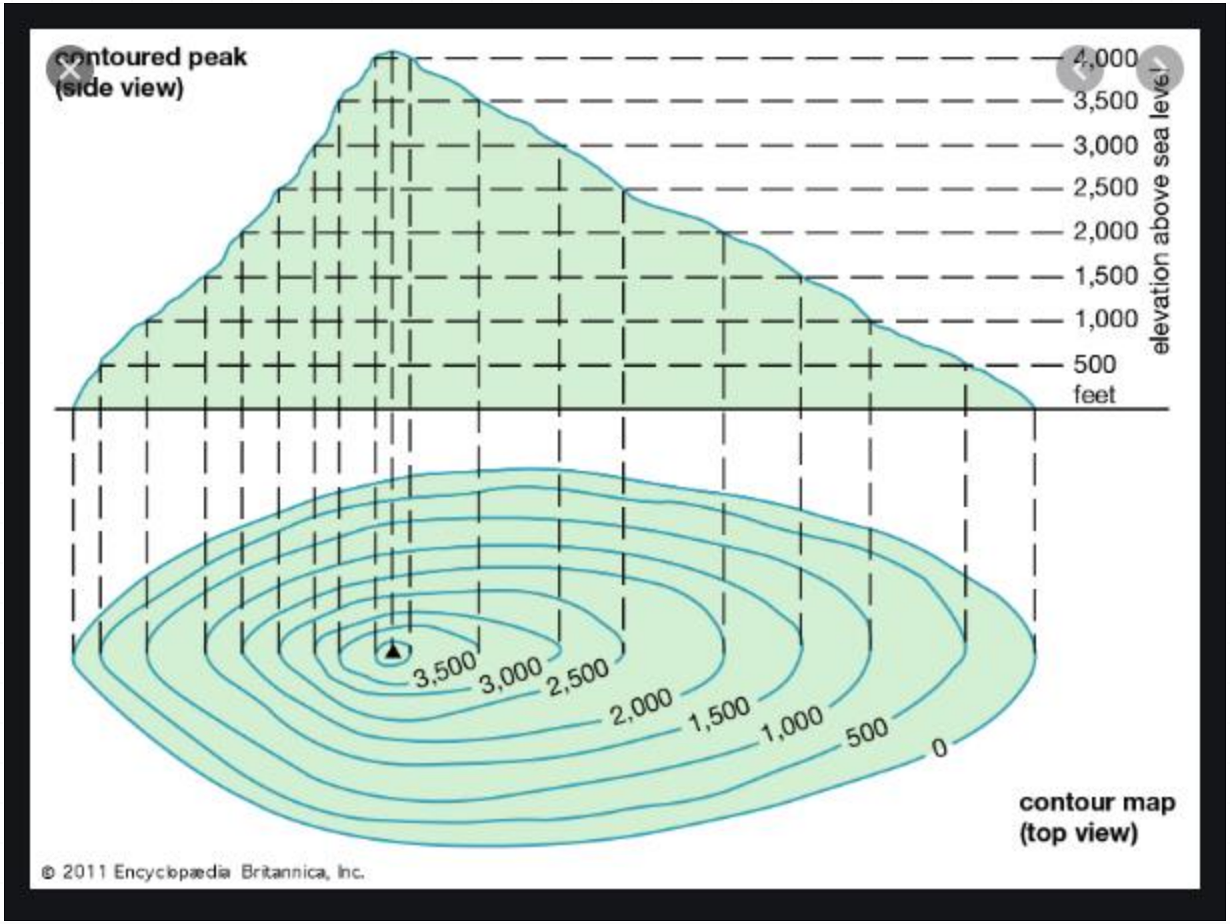
- Topographic surveys: used to determine the position and elevation of natural & manmade features. These features can then be drawn to scale on a plan or map.

- All topographic surveys are tied into both horizontal (X & Y reference grid) and vertical controls (Bench Mark).

Topographical surveys will help you to make plans or maps of an area that show the main physical features on the ground, such as rivers, lakes, reservoirs, roads, forests or large rocks; or the various features, such as ponds, dams, dikes, drainage ditches or sources of water;
the difference in height between land forms, such as valleys, plains, hills or slopes. These differences are called the vertical relief.



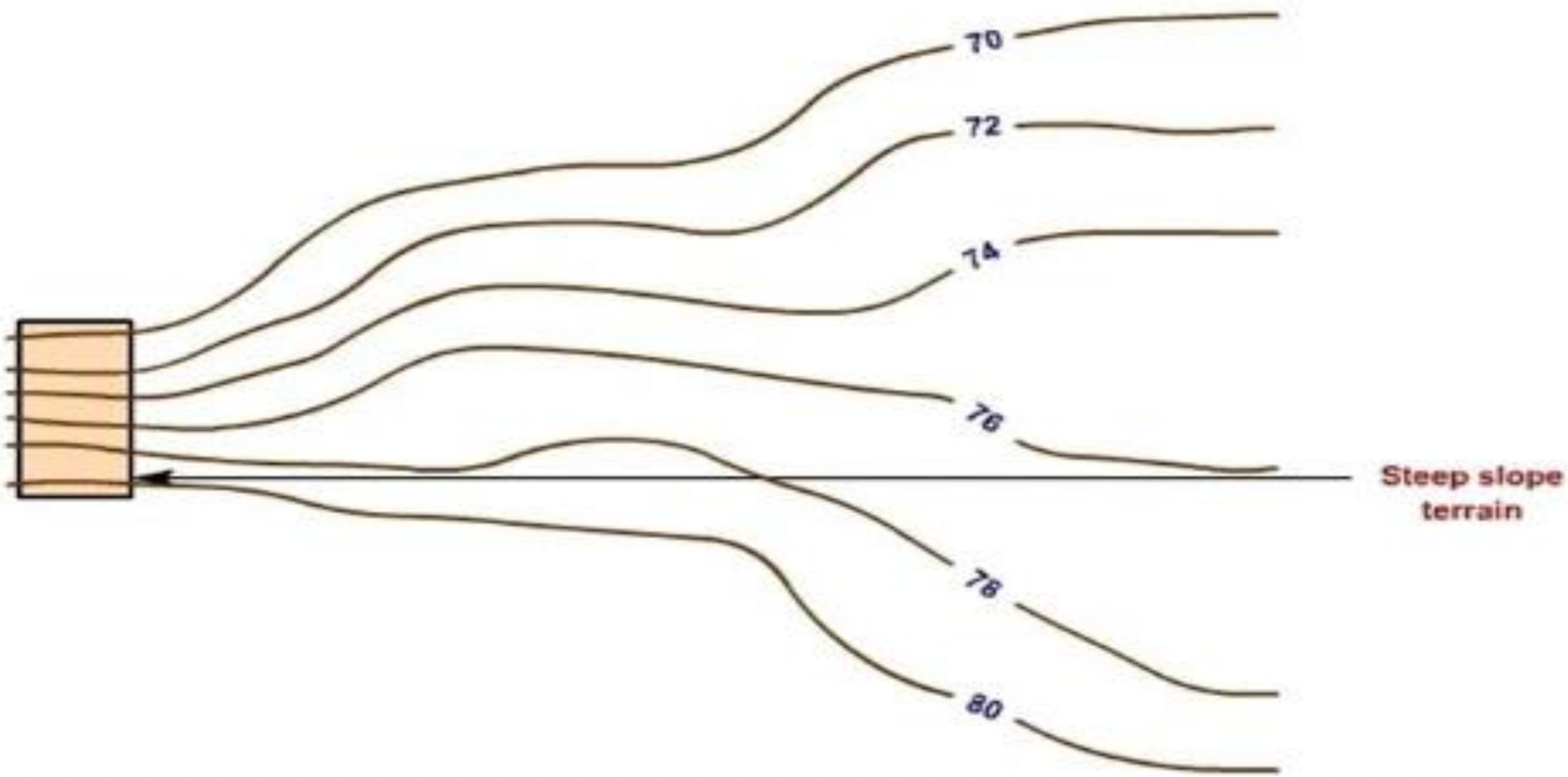
Contour line: a line on a map representing an imaginary line on the land surface, all points of which are at the same elevation above a datum plane, usually mean sea level.



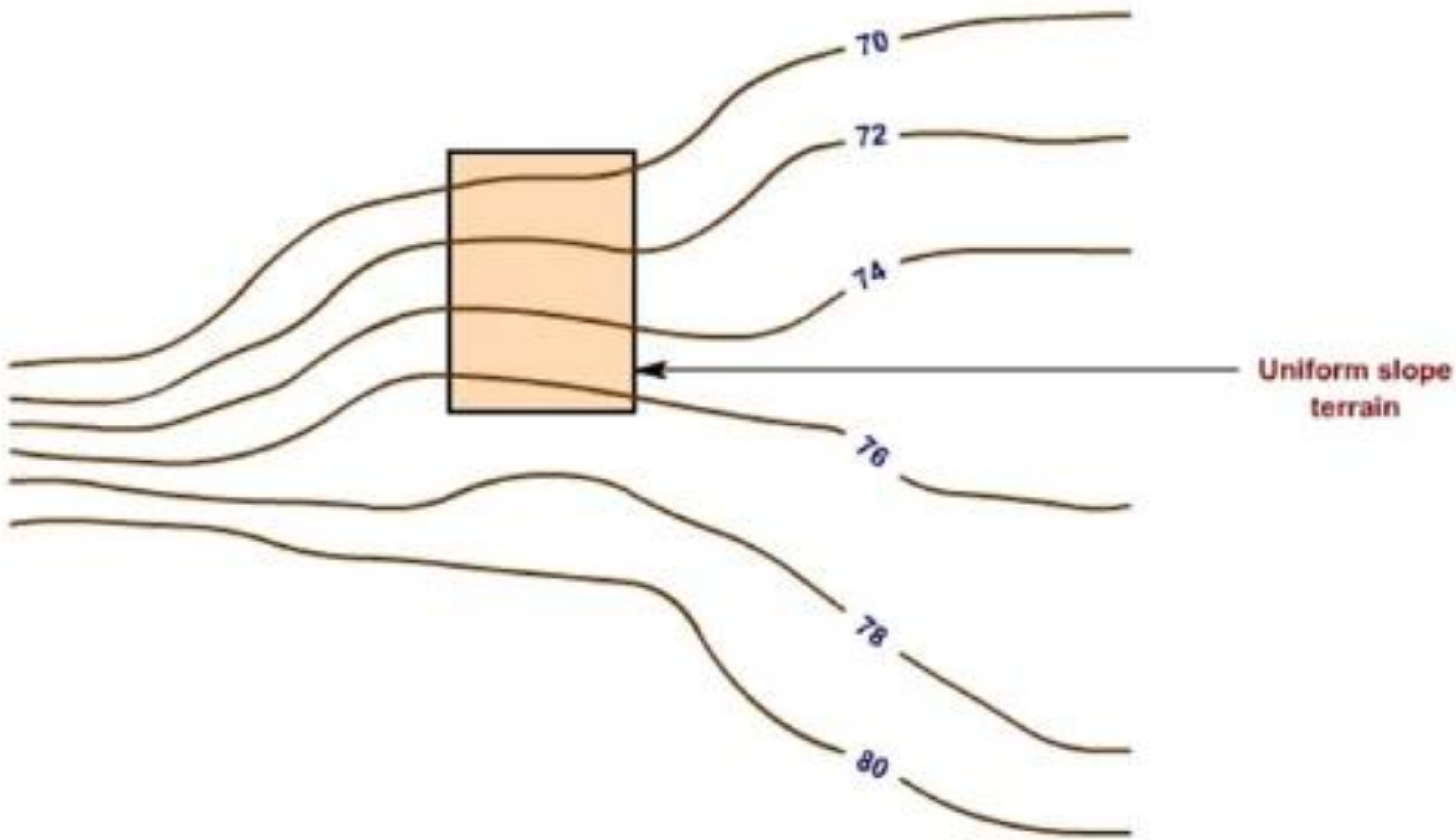
CHARACTERISTICS OF CONTOURS

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

1. Contour lines must close, not necessarily in the limits of the plan.
2. The horizontal distance between any two contour lines indicates the amount of slope and varies inversely on the amount of slope.
3. Widely spaced contour indicates flat surface.
4. Closely spaced contour indicates steep slope ground.



5. Equally spaced contour indicates uniform slope.

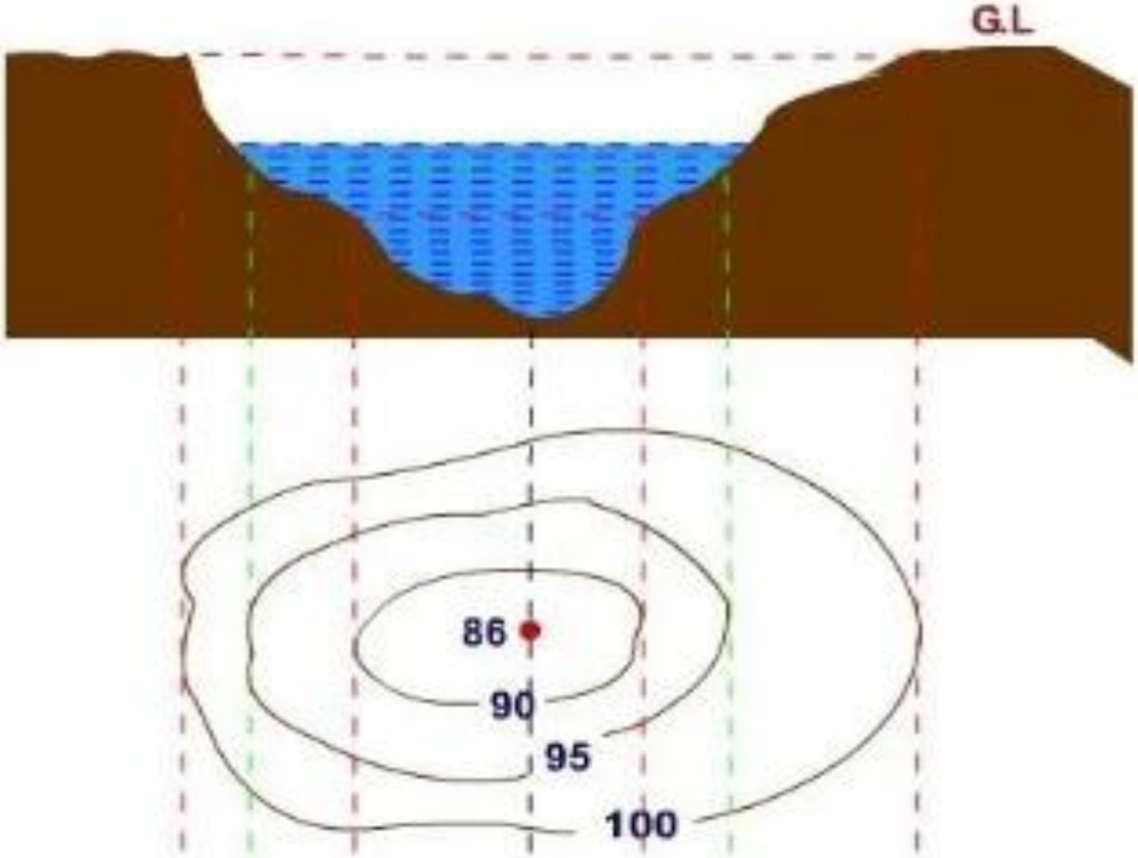


Contours of Terrain having different types of slope

contour showing uniform slope terrain

6. Irregular contours indicate uneven surface.

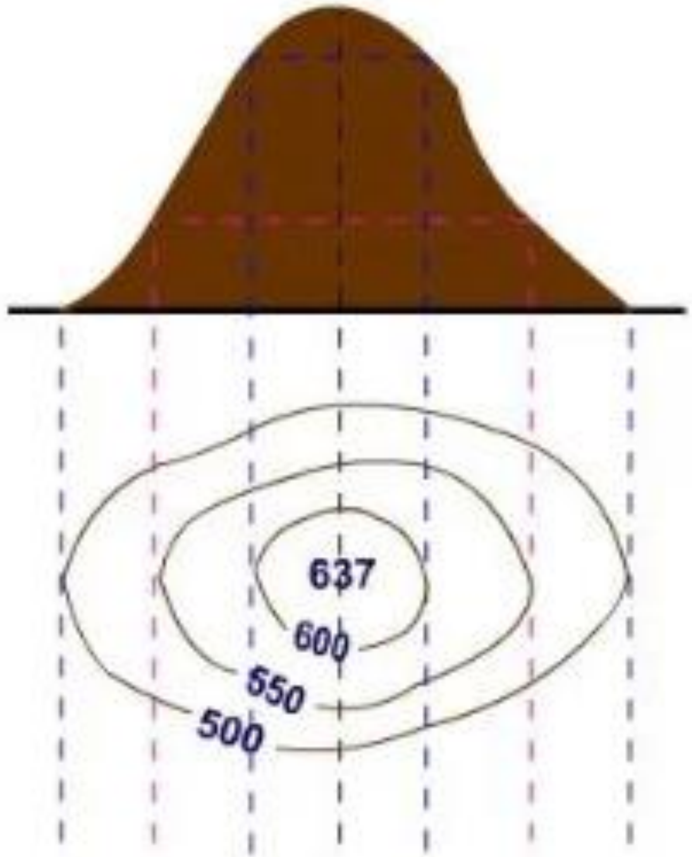
7. Approximately concentric closed contours with decreasing values towards centre indicate a pond.



Pond and its contour

Pond and its contour

8. Approximately concentric closed contours with increasing values towards centre indicate hills.

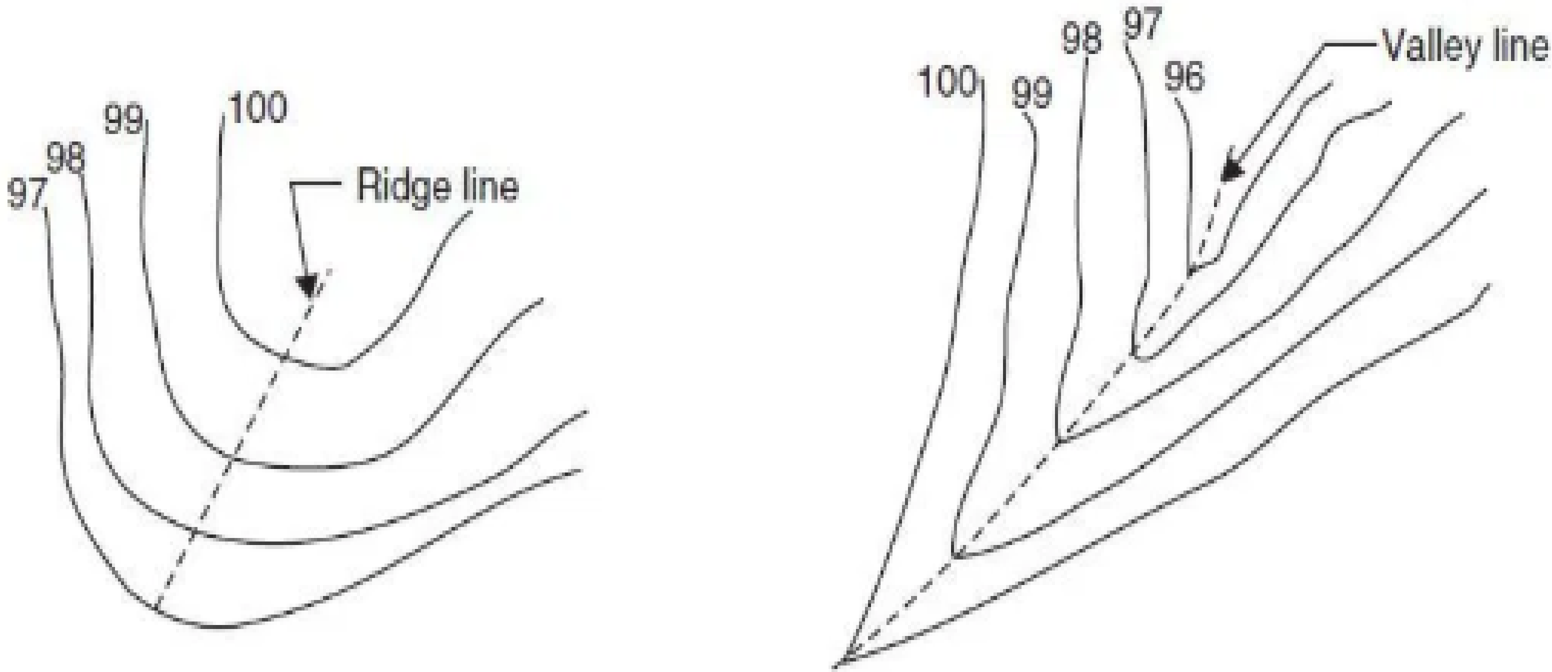


Hill and its contour

Hill and its contour

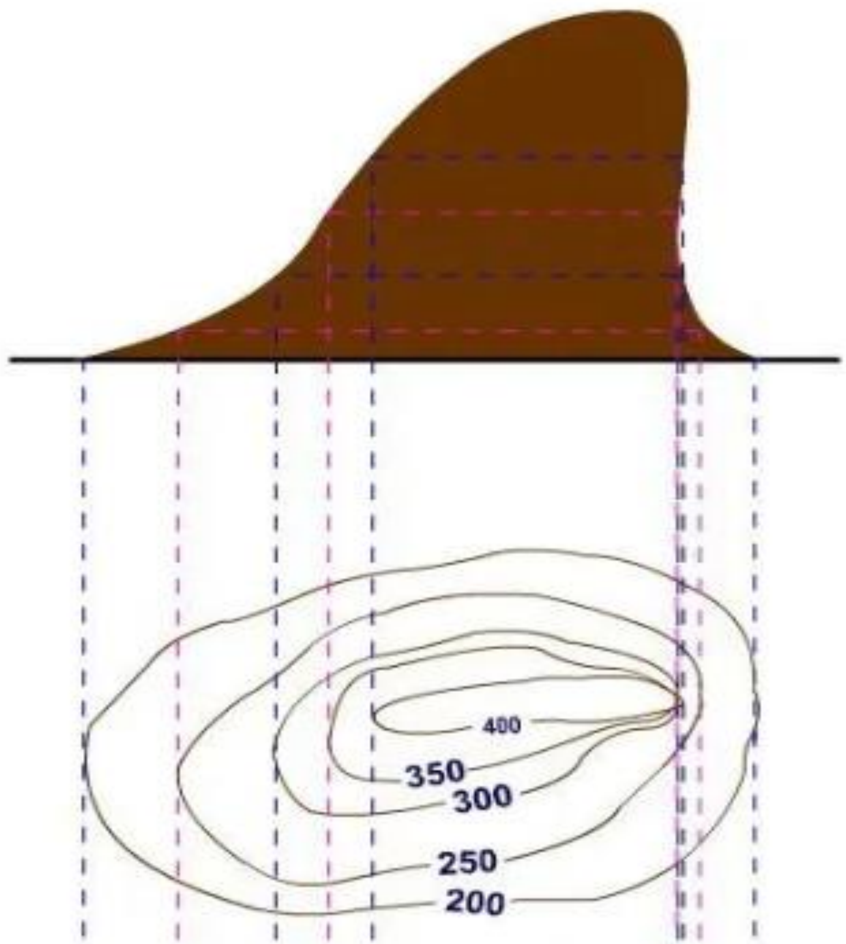
9. Contour lines with U-shape with convexity towards lower ground indicate ridge.

10. Contour lines with V-shaped with convexity towards higher ground indicate valley.



contour showing ridge line and valley line

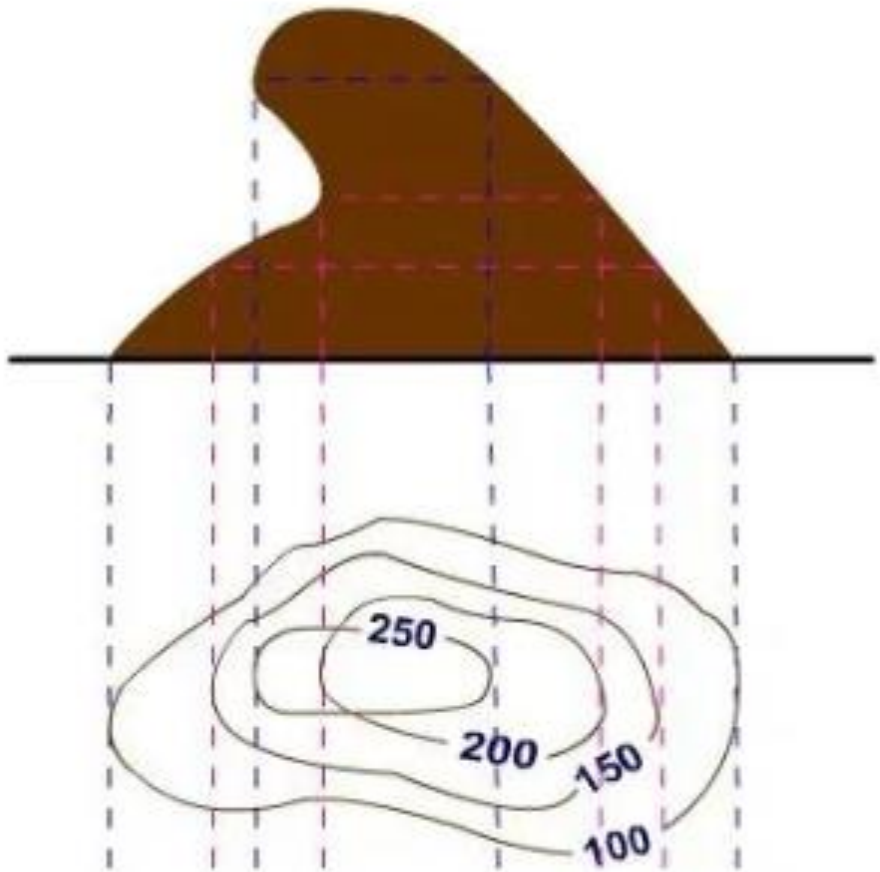
11. Contour lines generally do not meet or intersect each other. If contour lines are meeting in some portion, it shows existence of a vertical cliff.



Vertical cliff and its contours

Vertical cliff and its contour

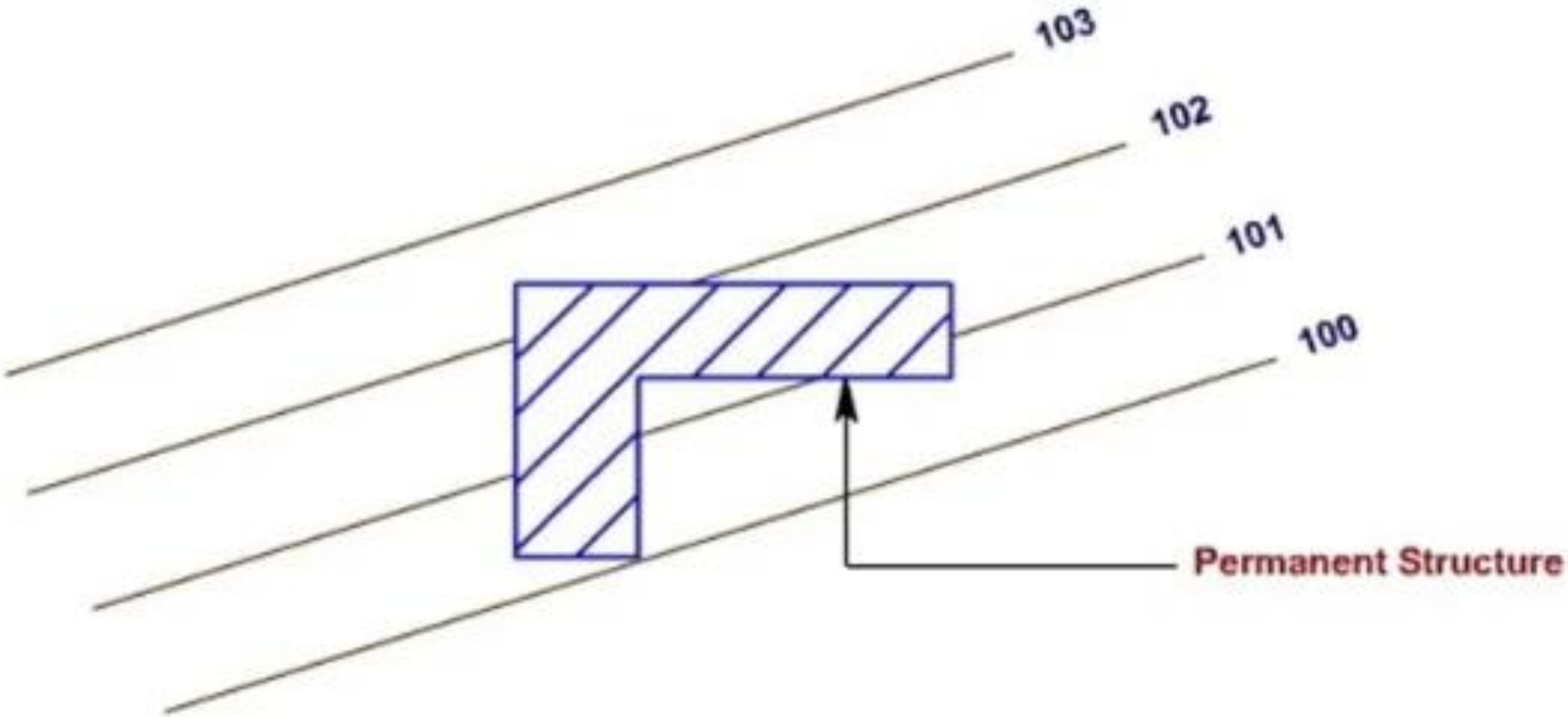
12. Contours of different elevations cannot cross each other. If contour lines cross each other, it shows existence of overhanging cliffs or a cave.



Overhanging cliff and its contour

overhanging cliff and its contour

13. The steepest slope of terrain at any point on a contour is represented along the normal of the contour at that point.
14. Contours do not pass through permanent structures such as buildings.



Contours across permanent structure

contour across a permanent structure

⇒ **Scale and precision**

1 : 100 ⇒ 1cm on map = 1 m on land
or
1" on map = 100" on land

Large scale	Intermediate	Small scale
1 : 100		
1 : 200	1 : 2,000	1 : 20,000
1 : 500	1 : 5,000	1 : 50,000
1 : 1000	1 : 10,000	1 : 100,000
		1 : 200,000
		1 : 500,000
		1 : 1,000,000
		1 cm = 10 Km

⇒ Field precision should be compatible with possible on map plotting precision at the designated map scale.

Example 1.

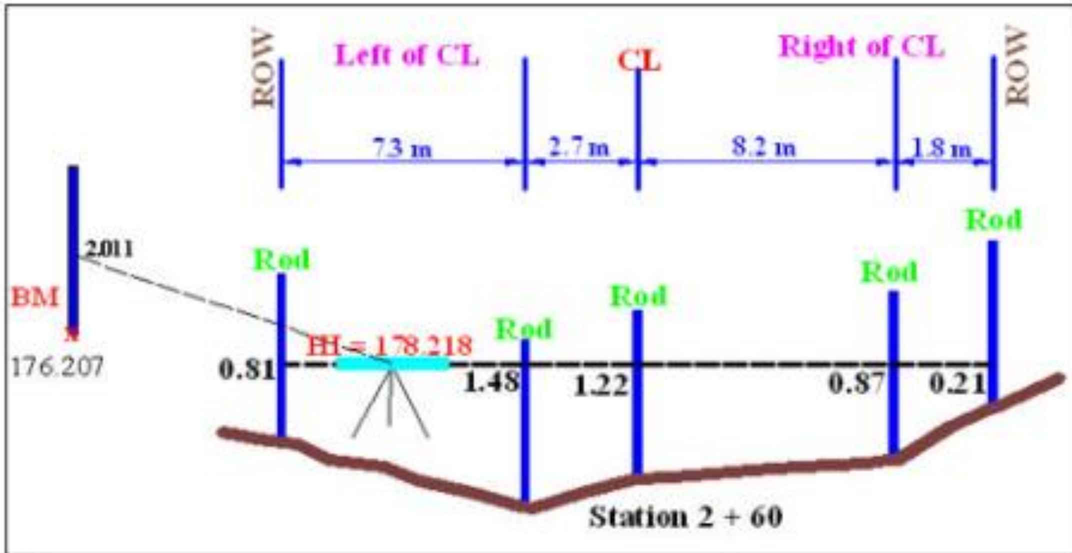
If points can be plotted to the closest 0.5 mm at scale 1:500

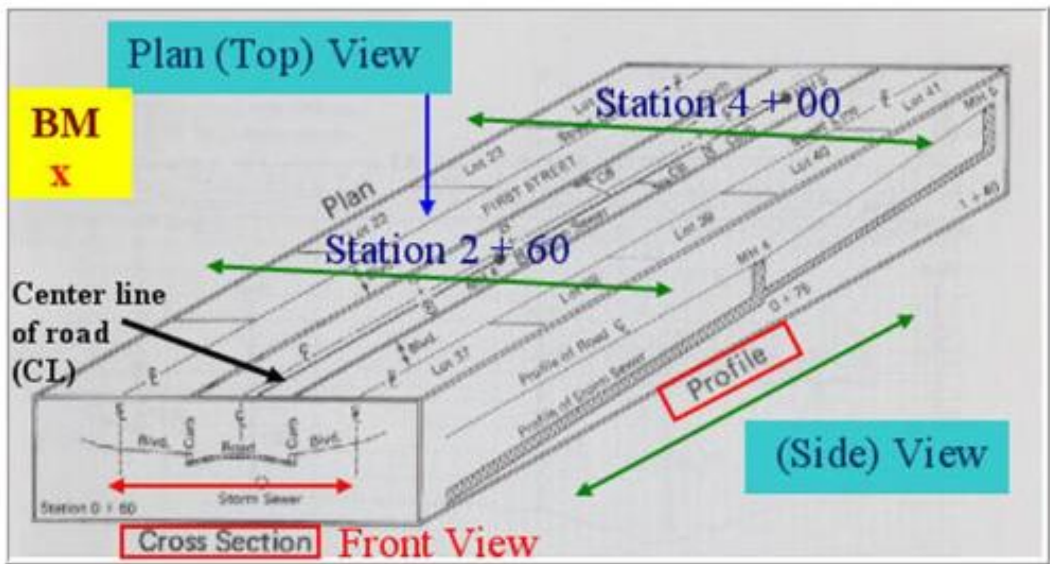
Then: Required field precision = 0.5 * 500 mm

= 250 mm = <u>0.25 m</u>

Cross Sections & Profiles

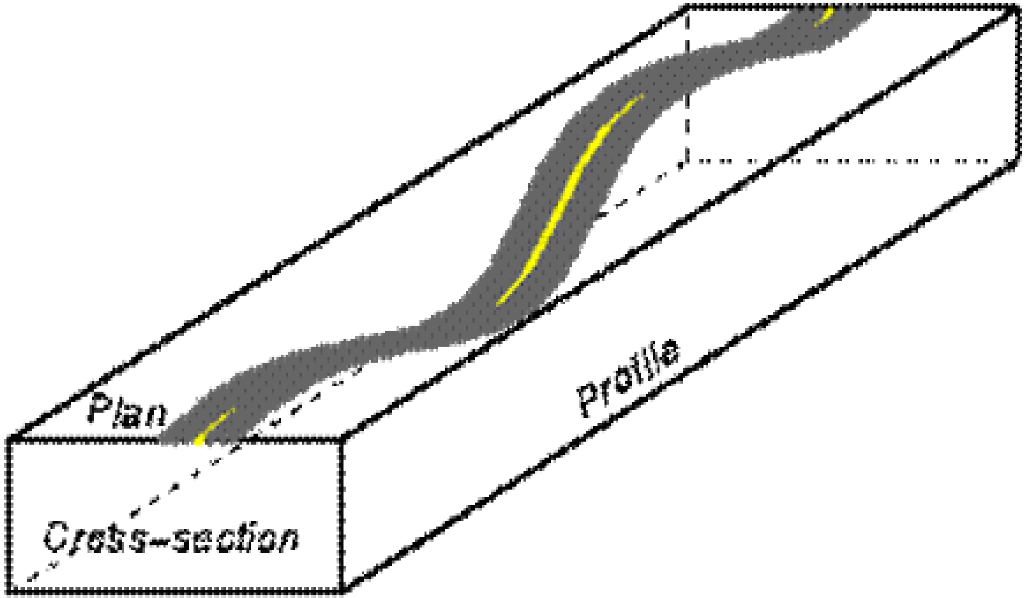
⇒ **Cross section:** a series of elevations taken at right angles to a baseline at <u>specific stations</u>.

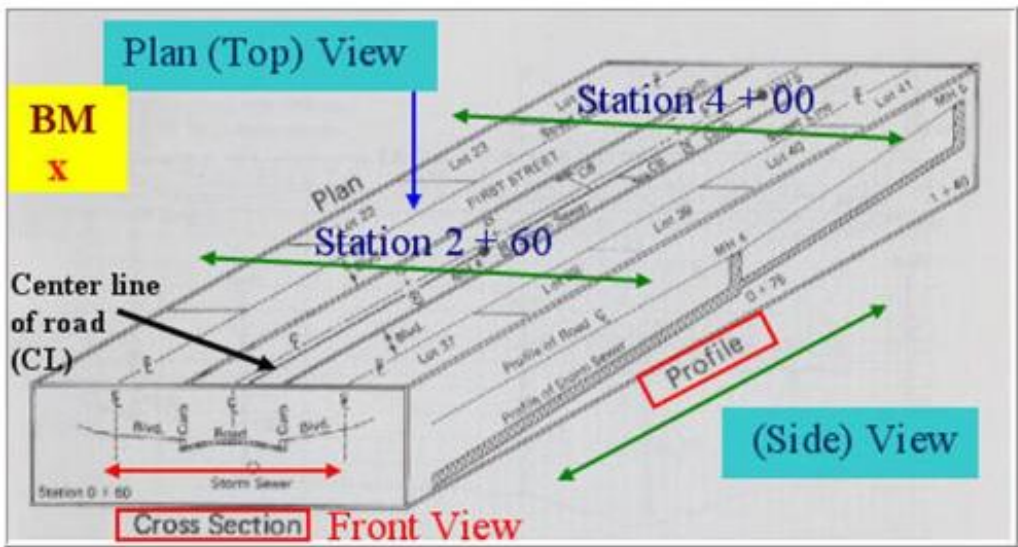




Cross Sections & Profiles

Profiles: a series of elevations taken along a baseline at some specified repetitive station interval.

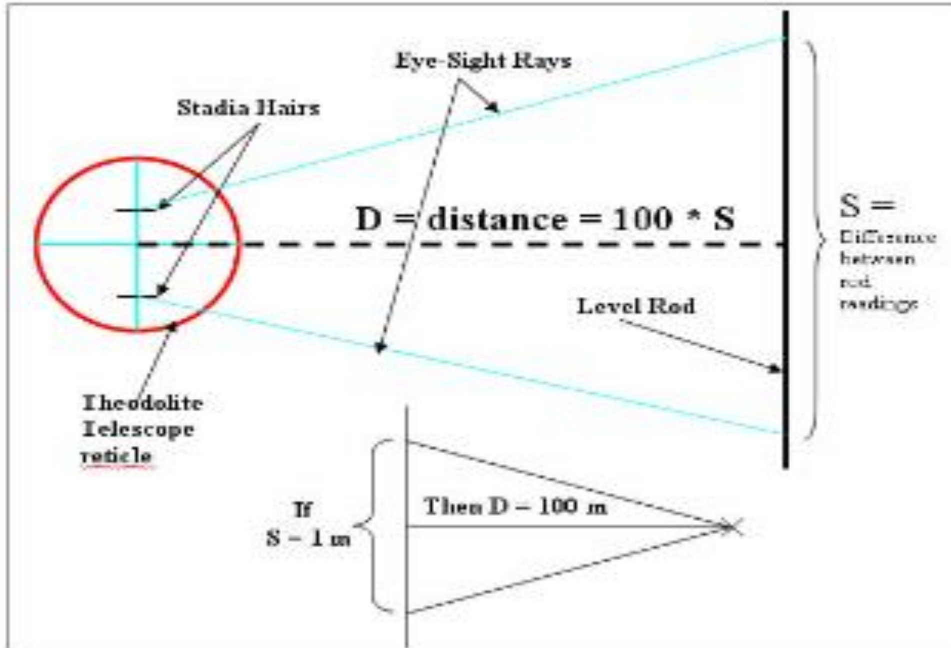




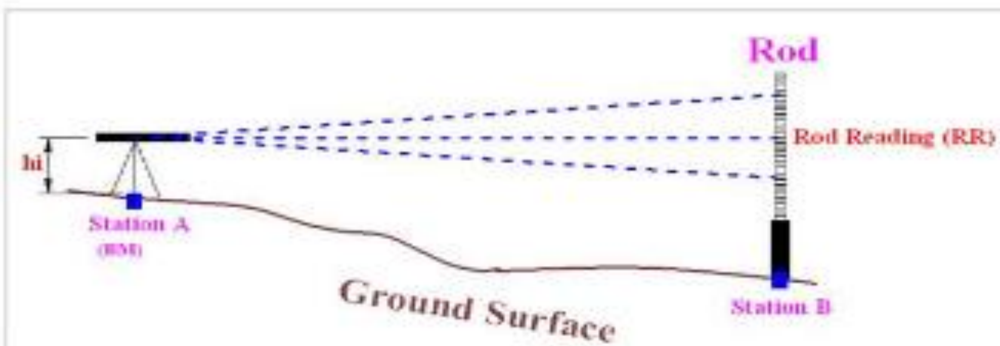
Stadia Principles

- Tachometry technique (uses trigonometry calculation) to measure distances.
- Used in topographic surveys where accuracy is around 1/400.
- Uses the horizontal marks on the theodolite or level cross-hair.
- Stadia hairs are positioned in the reticle so that, if a rod is held 100m away from instrument, the difference between upper and lower stadia hairs readings on a level rod is 1m.

Stadia Principles



⇒ **Horizontal distance (D) = Rod interval (S) * 100**

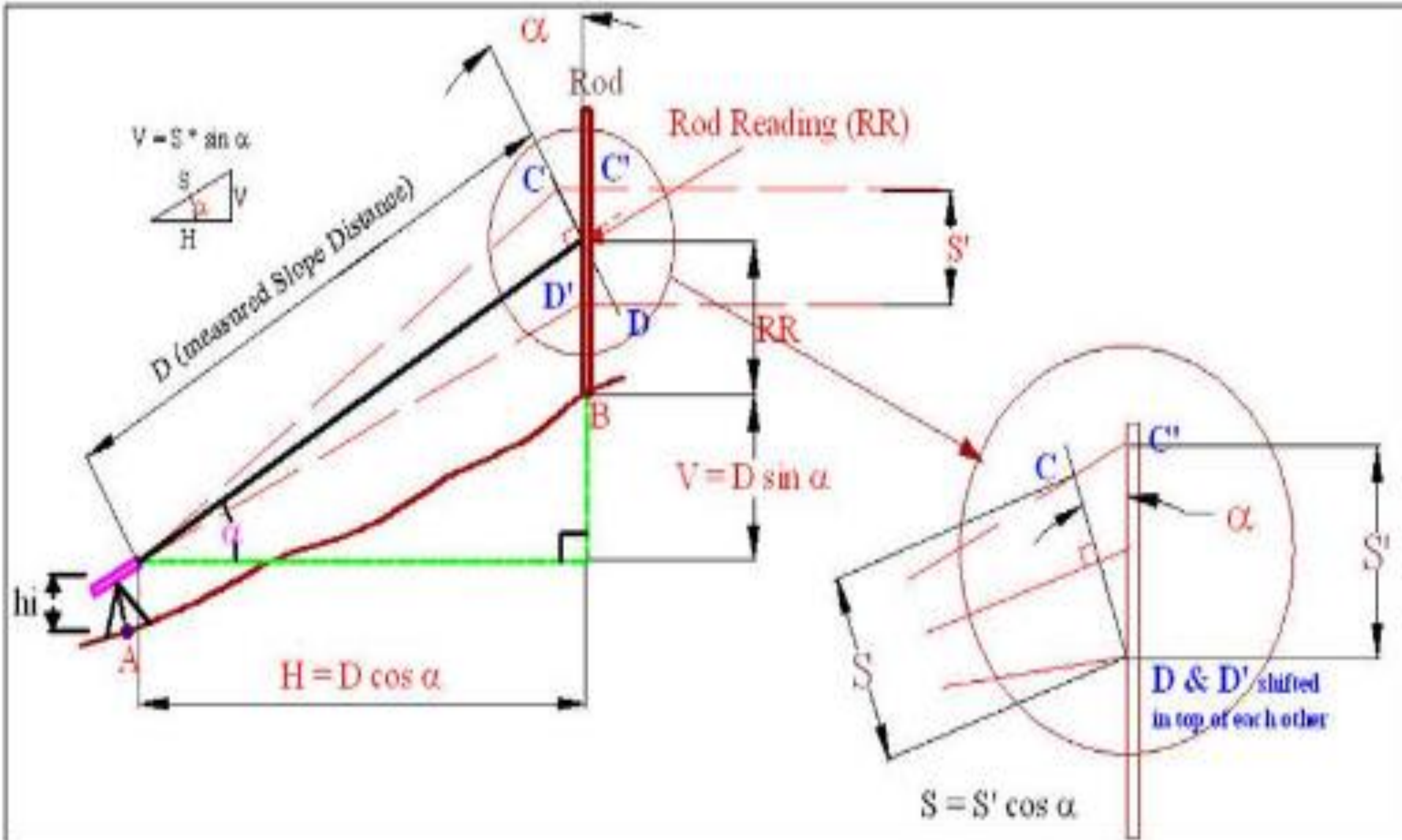


- **If level or theodolite with leveled telescope in addition to the rod are used**
  - **Horizontal distance (D) = Rod interval(S) * 100**
  - **Elevation of B = Elevation of A + hi – RR**

Inclined Stadia Measurements

- **If the stadia principle is applied when the rod is on a hilly area, the telescope will be inclined leading to error in the read interval.**
- **The rod interval of a sloped sighting must be reduced to what the interval would have been if the line of sight had been perpendicular to the rod.**
- **The read interval is S', while the corrected interval is S.**

$$S = S' \cos \alpha$$


α
Rod
Rod Reading (RR)
V = S * sin α
S
V
H
α
C'
C"
D (measured Slope Distance)
S'
D'
D
RR
B
V = D sin α
α
hi
A
H = D cos α
C
C"
α
S'
S
D & D' shifted in top of each other
S = S' cos α

- $D = 100\,S$
- $S = S' \cos \alpha$
- $D = 100\,S' \cos \alpha$
- $H = D \cos \alpha$

$\therefore\ H = 100\,S' \cos^2 \alpha$

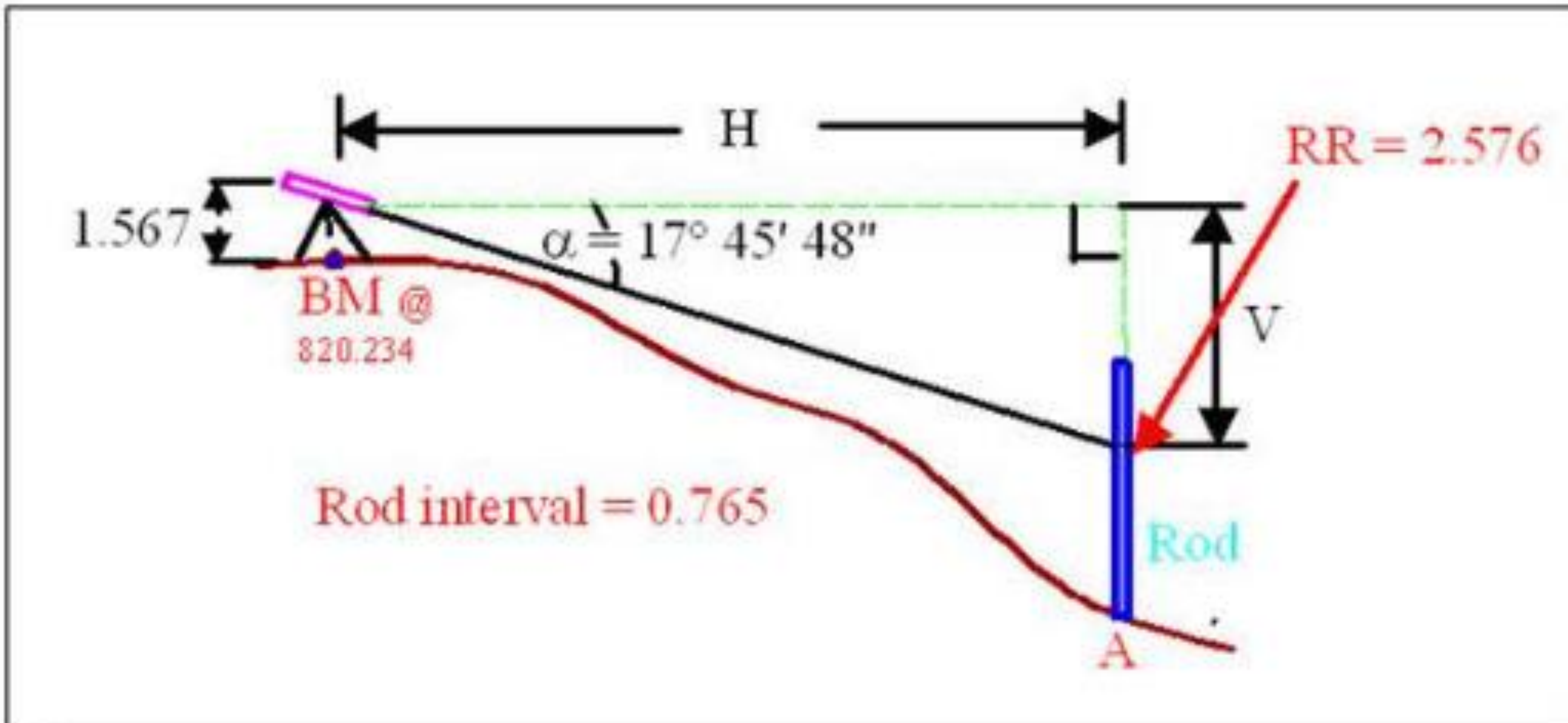
- $V = D \sin \alpha$
- $D = 100\,S' \cos \alpha$

$\therefore\ V = 100\,S' \cos \alpha \sin \alpha$

$\therefore$ Elevation of B = Elevation of A + hi + V – RR

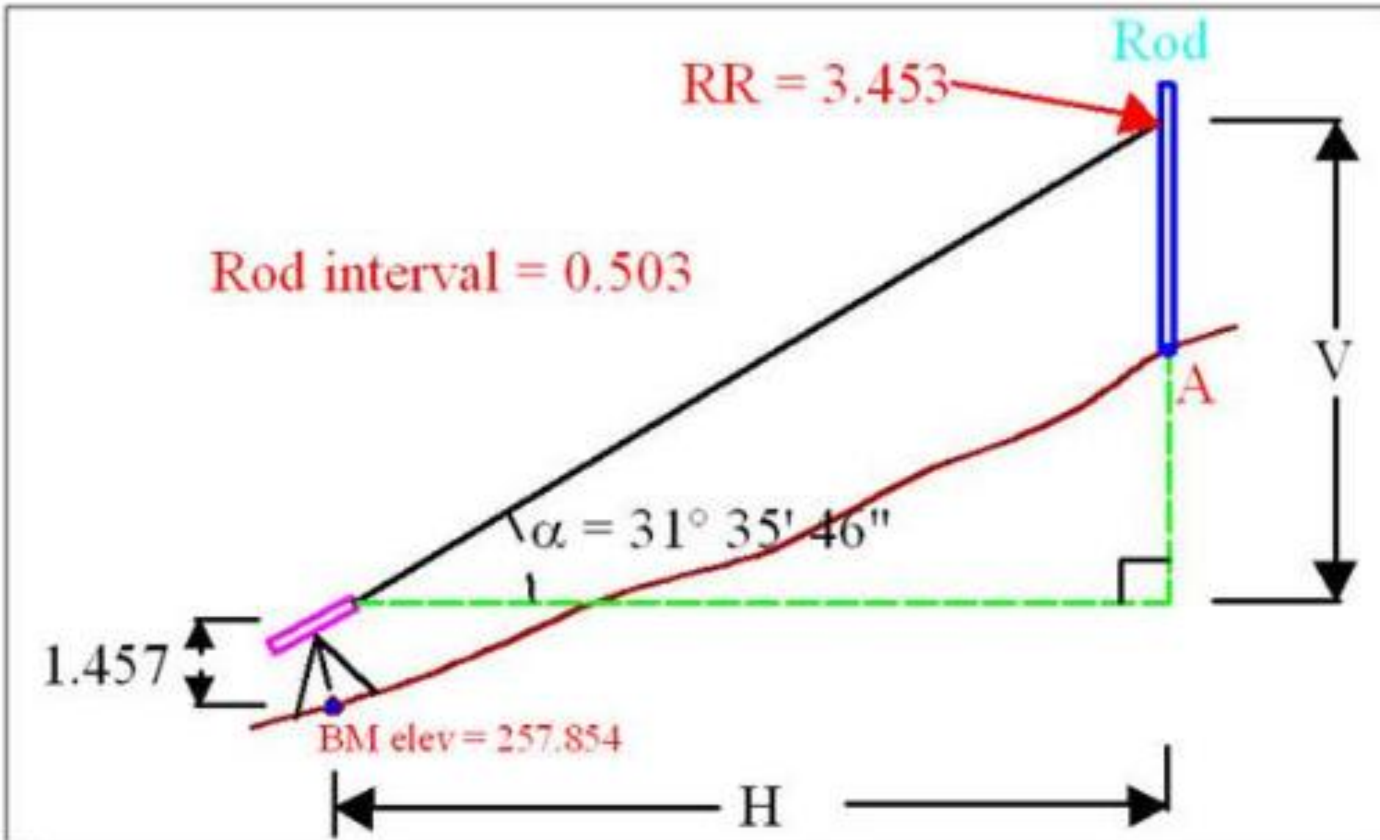
The vertical distance (V) should be taken with its sign.
i.e., if A is lower than B ⇒ the angle α is an angle of inclination (+ve) leading to (+ve) V.
i.e., if A is higher than B ⇒ the angle α is an angle of depression (-ve) leading to (-ve) V.

Q 1. Calculate elevation of point A in the following figure:

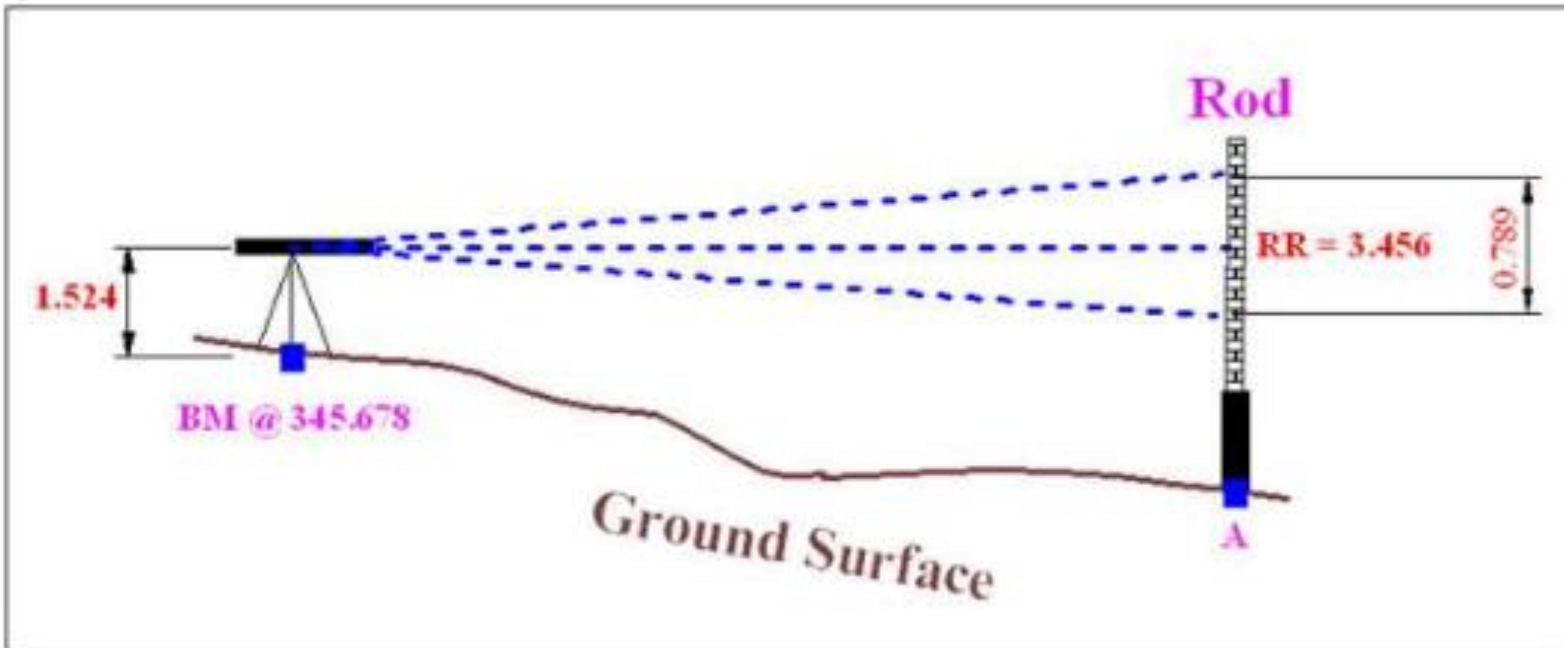


Prof. TA... ...USAN

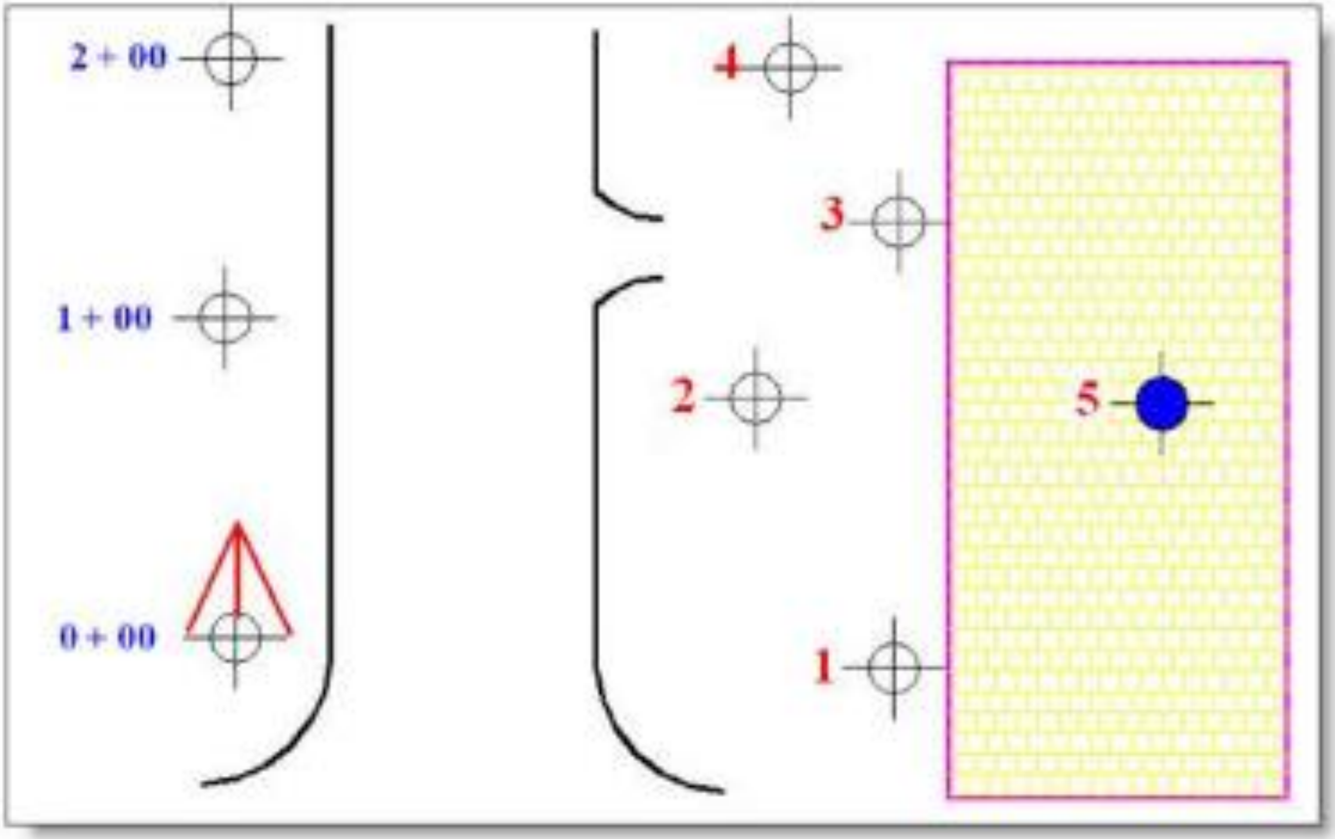
Q 2. Calculate elevation of point A in the following figure:



Q 3. Calculate elevation of point A in the following figure:



Stadia Field Practice



Q1. If the coordinates of Station 0+00 are (245.47, 348.57) find the (x, y, z) coordinates of all the stations and points shown in the above figure.

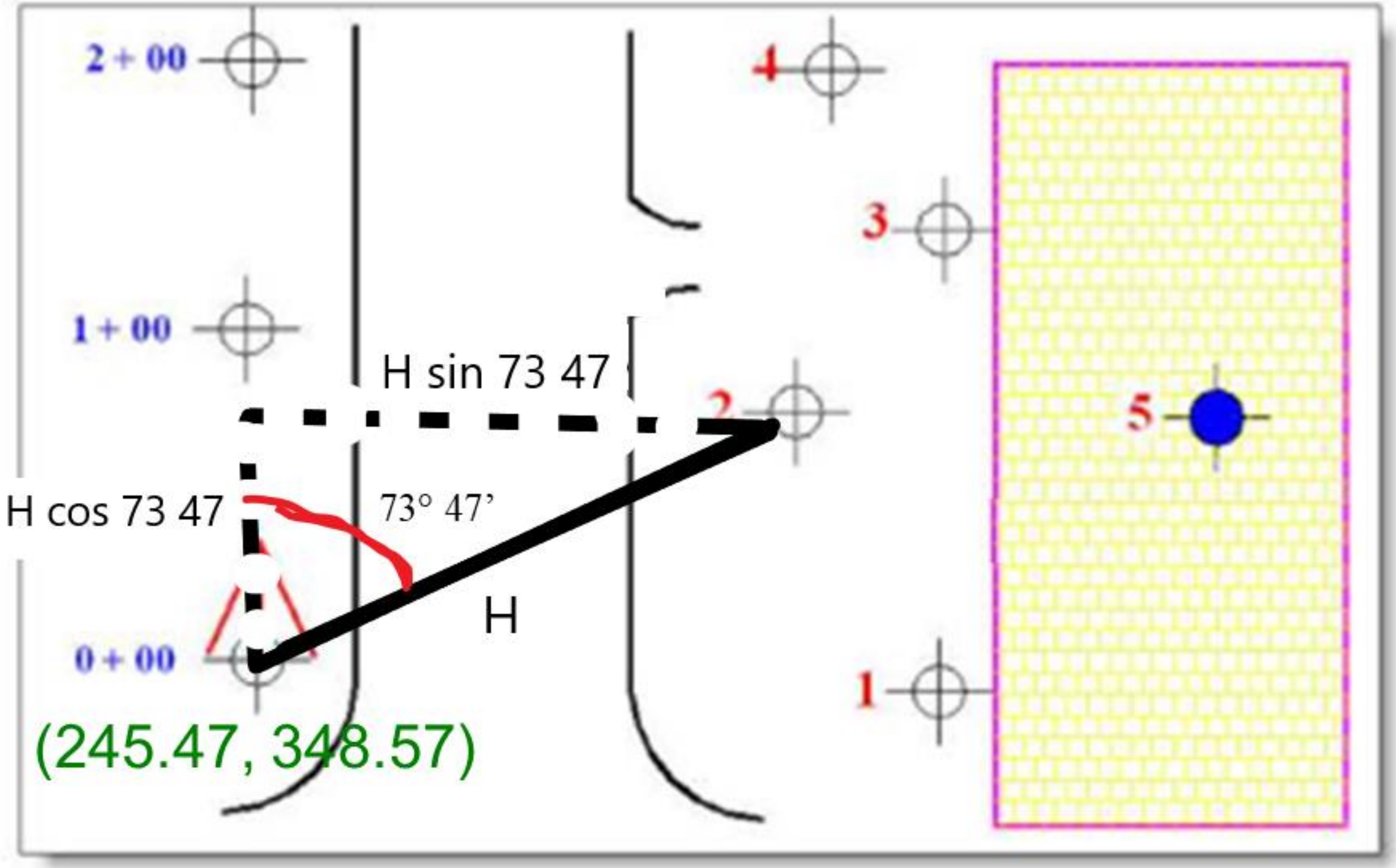
Station	Hor. Angle	Rod Interval	Ver. Angle	Hor. distance	RR	Elev. Difference	Elevation
0 + 00	Theodolite at this Station & hi = 1.55						222.32
1 + 00	0° 30’	1.002	+ 5° 36’	99.2*	1.55	9.73**	232.05 ♦
2 + 00	5° 20’	0.401	- 1° 24’	40.1	1.55	- 0.98	221.34
①	93° 25’	0.723	+ 5° 38’	71.6	1.55	7.06	229.38
②	73° 47’	1.245	- 7° 54’	122.1	2.03	- 16.95	204.89
③	68° 32’	2.075	- 12° 24’	197.9	1.32	- 43.52	179.03
④	35° 20’	0.224	0° 00’	22.4	2.22	0.00	221.65
⑤	80° 30’	3.123	+ 42° 24’	170.3	1.87	155.51	377.51

* $H = 100\ S' \cos^2 \alpha = 100 * 1.002 * (\cos(+5° 36'))^2 = 99.2$

** $V = 100\ S' \cos \alpha \ \sin \alpha = 100 * 1.002 * \cos(+5° 36') * \sin(+5° 36') = 9.73$

♦ Elevation of B = Elevation of A + hi + V – RR = 222.32 + 1.55 + 9.73 – 1.55 = 232.05

In some table formats the RR column is omitted and if RR is equal to hi the RR value is not mentioned. If RR is not equal to hi the RR value is placed in the same cell of the Ver. angle value. (See textbook for an example).



2 + 00
1 + 00
0 + 00
4
3
2
5
1
H sin 73 47
H cos 73 47
73° 47'
H
(245.47, 348.57)

Prof. TALEB AL-ROUSAN

Surveying

2104011365

Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University

Prof. TALEB AL-ROUSAN

Chapter 8
Drafting & Computation

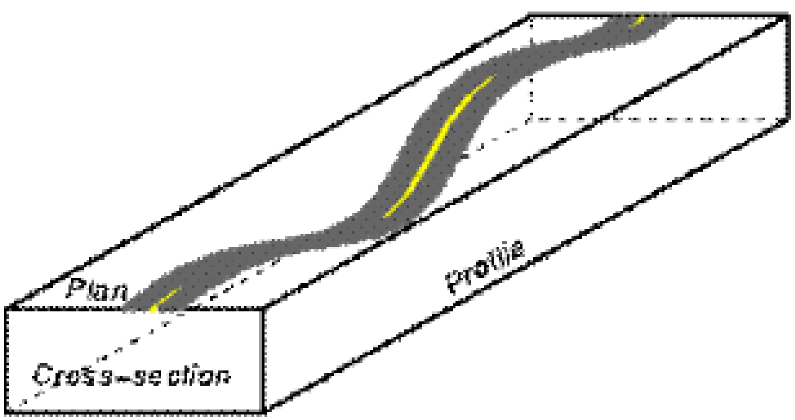
Drafting Profiles

Profiles: Establish ground elevation along a defined route.

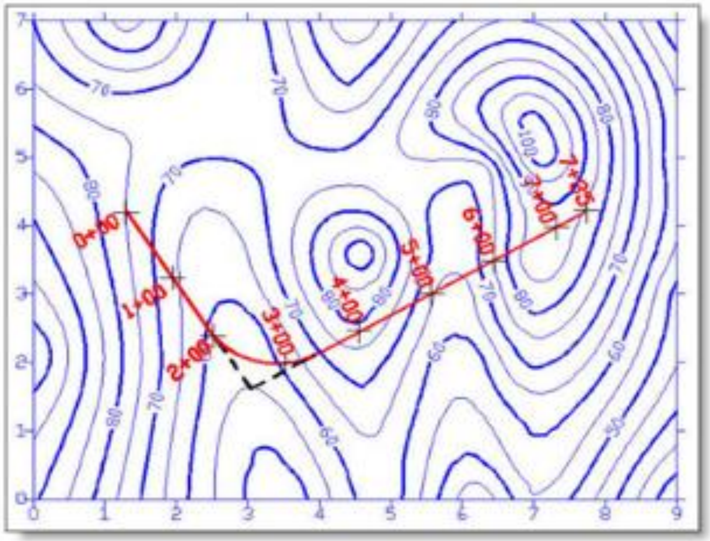
- Can be directly surveyed.

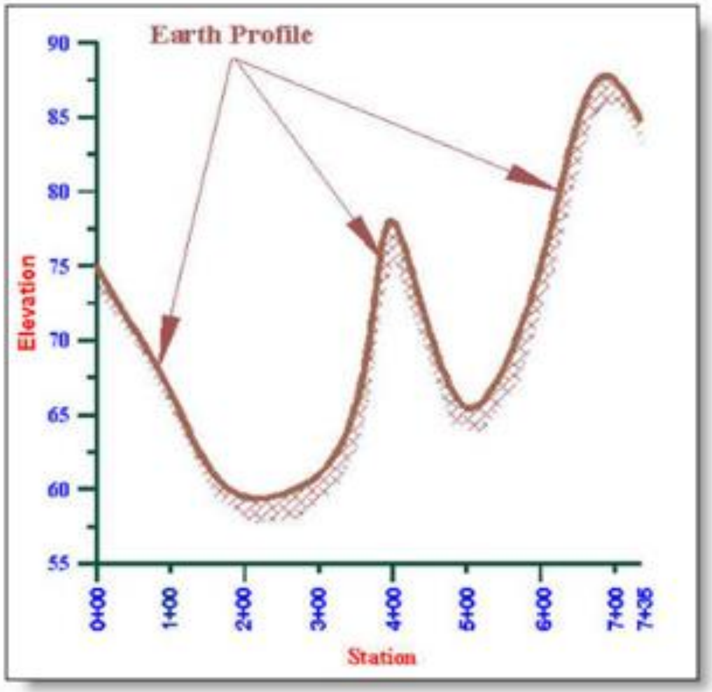
Example:

Elevation of road-center line taken at specific intervals.



- Can be taken from contour drawing.

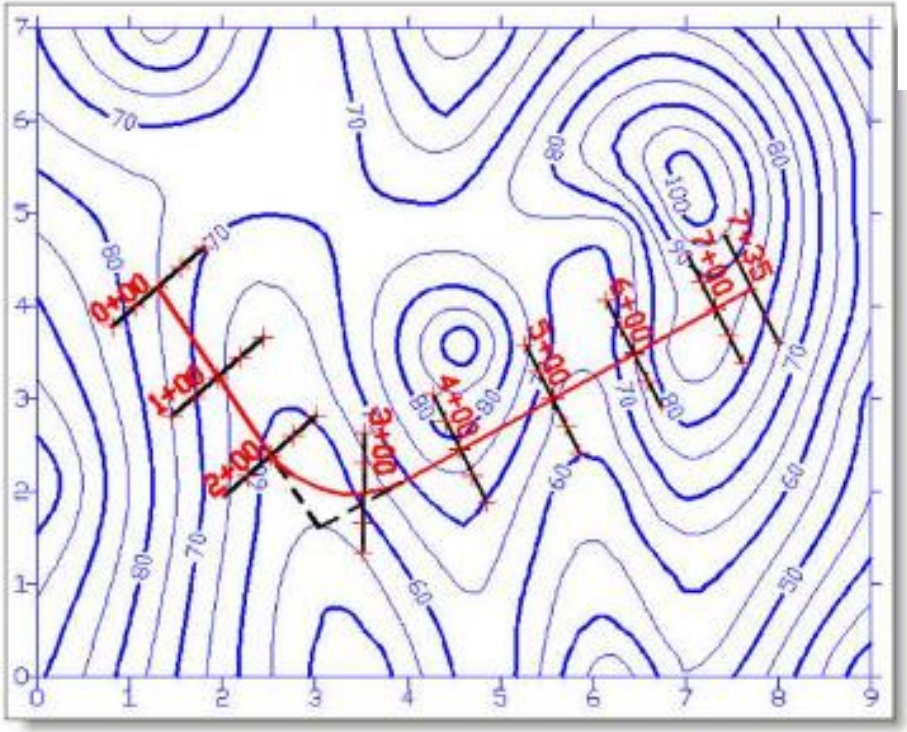




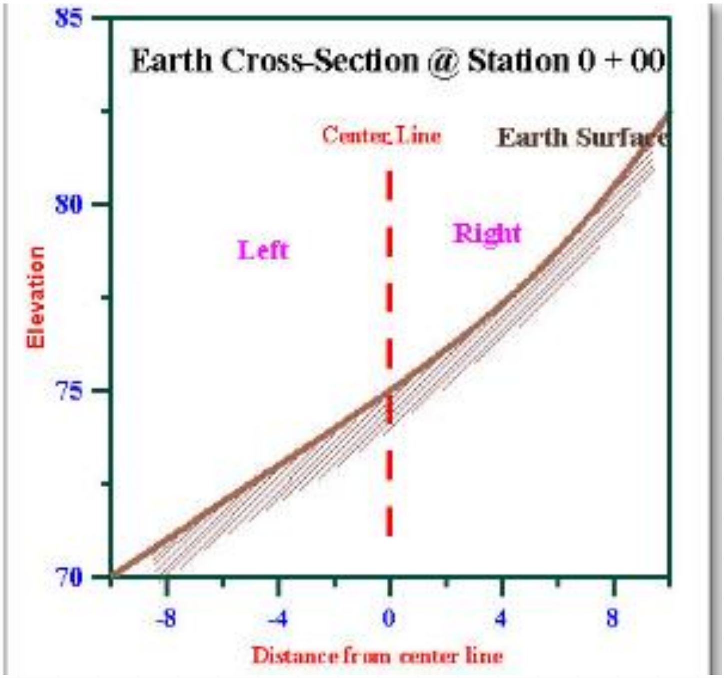
Drafting Cross Sections

Cross section: ground elevation at right angles to a proposed route.

- **Used for determining quantities of cut and fill.**
- **Can be developed from a contour plan.**



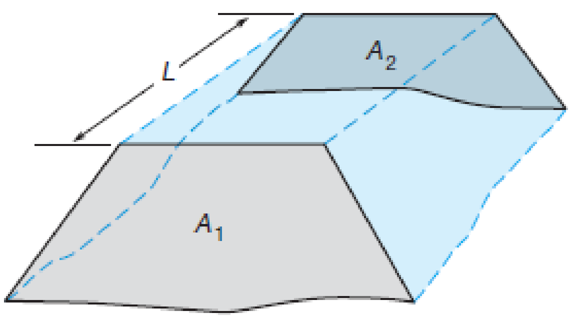
The cross-section at station 0+00 will appear like this:

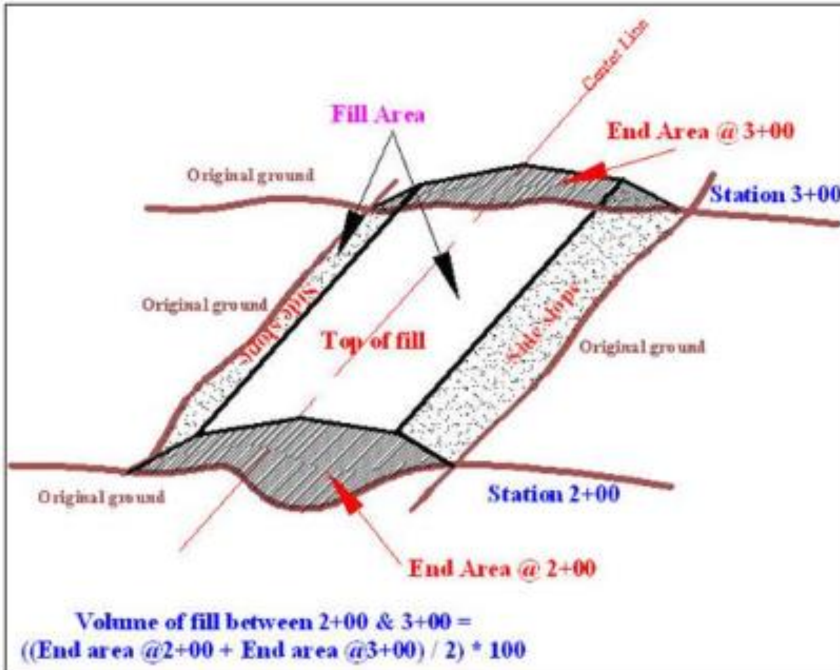


Volume Calculations

End Area Method

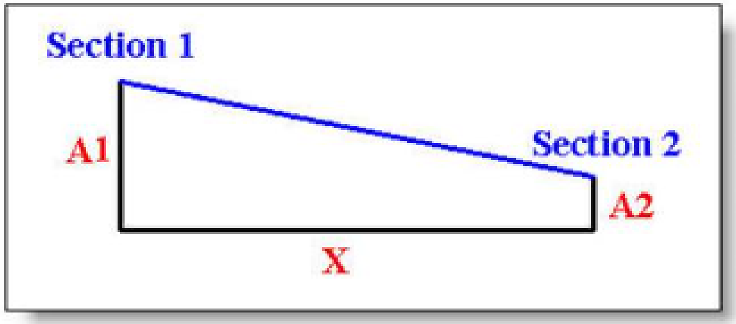
- Not Precise
- Cross section area = A_1
- Cross section area = A_2
- Average area = $(A_1 + A_2)/2$
- Volume = L * average area.





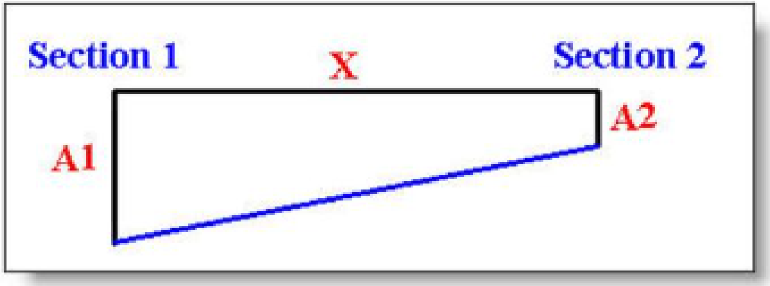
Volume Calculation / End Area Method

- If successive cross sections are both cut sections ⇒



$$Volume\ of\ cut\ between\ Sec.1\ \&\ Sec.2 = \frac{A1 + A2}{2} * X \qquad (1)$$

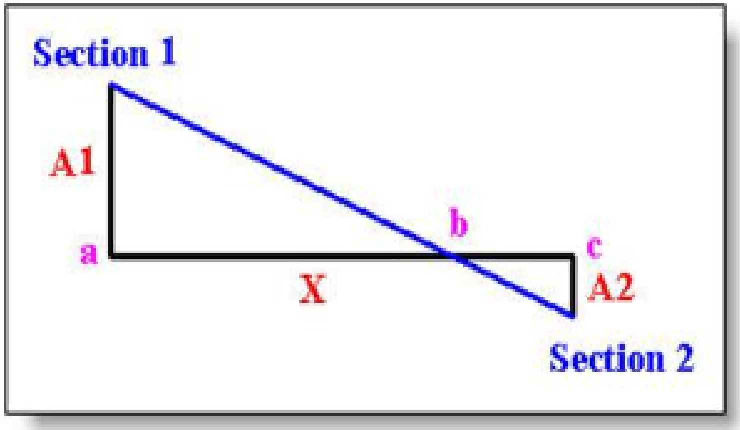
- If successive cross sections are both fill sections ⇒



$$Volume\ of\ fill\ between\ Sec.1\ \&\ Sec.2 = \frac{A1 + A2}{2} * X \qquad (2)$$

Volume Calculation/ End Area Method

- If one of the successive cross sections is fill and the other one is cut ⇒



Get length of ab from:

$$\frac{A1}{ab} = \frac{A2}{X - ab} \qquad (3)$$

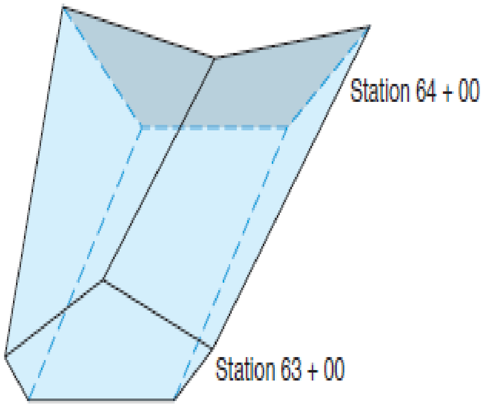
Volume of cut = 1/2 * A1 * ab (4)

Volume of fill = 1/2 * A2 * bc (5)

Volume Calculation/Prismoidal Formula

- **Prismoid: many-sided body with two bases that are polygons in parallel planes.**
- **More precise than end area volume**
- **Good when sections are changing from cut to fill**
- **Usually used for expensive cut and fill operations**

$$V = L\,\frac{(A_1 + 4A_m + A_2)}{6}$$



A_1: Face area

A_m: Middle area (at ½ L)

A_2: Back area

L: Distance between A_1 & A_2

The prismoidal formula generally gives a volume SMALLER than that found by the average-end-area formula.

METHODS OF MEASURING AREA

Both field and map measurements are used to determine area.

Field measurement methods are the more accurate and include:

1. division of the tract into simple figures (triangles, rectangles, and trapezoids),
2. coordinates, and
3. double-meridian distances.
4. Offset from a straight line.

Methods of determining area from map measurements include:

1. Counting coordinate squares,
2. dividing the area into triangles, rectangles, or other regular geometric shapes,
3. digitizing coordinates, and
4. running a planimeter over the enclosing lines.

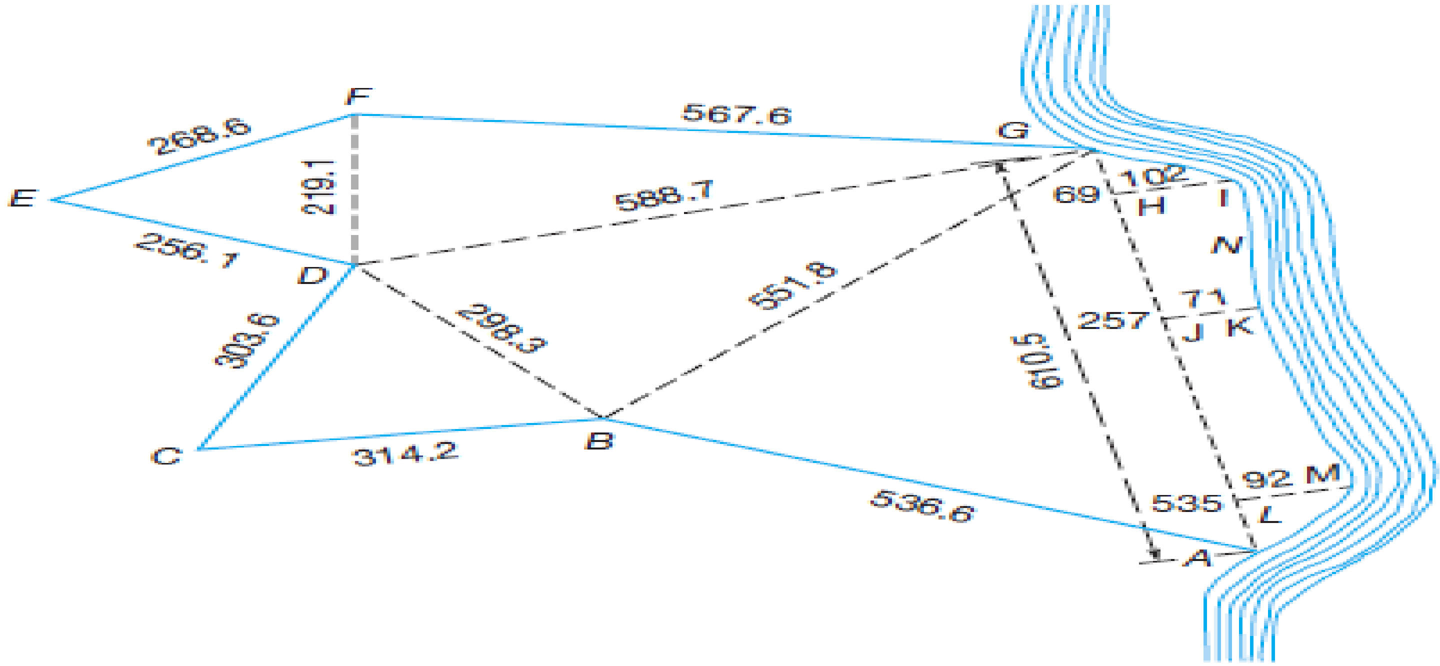
TABLE 12.1 APPROXIMATE AREA CONVERSION FACTORS

To Convert from	To	Multiply by
ft^2	m^2	$(12/39.37)^2 \approx 0.09291$
m^2	ft^2	$(39.37/12)^2 \approx 10.76364$
yd^2	m^2	$(36/39.37)^2 \approx 0.83615$
m^2	yd^2	$(39.37/36)^2 \approx 1.19596$
acres	hectares	$[39.37/(4.356 \times 12)]^2 \approx 2.47099$
hectares	acres	$(4.356 \times 12/39.37)^2 \approx 0.40470$

1. AREA BY DIVISION INTO SIMPLE FIGURES

A tract can usually be divided into simple geometric figures such as triangles, rectangles, or trapezoids. The sides and angles of these figures can be observed in the field and their individual areas calculated and totaled.

- **An example of a parcel subdivided into triangles is shown in the Figure below:**



Formulas for computing areas of rectangles and trapezoids are well known.

The area of a triangle whose lengths of sides are known can be computed by the Formula: Heron's Formula

$$area = \sqrt{s(s-a)(s-b)(s-c)}$$

where a, b, and c are the lengths of sides of the triangle and

$$s = \frac{1}{2}(a+b+c)$$

Another formula for the area of a triangle is:

$$area = \frac{1}{2} ab \sin C$$

where C is the angle included between sides a and b.

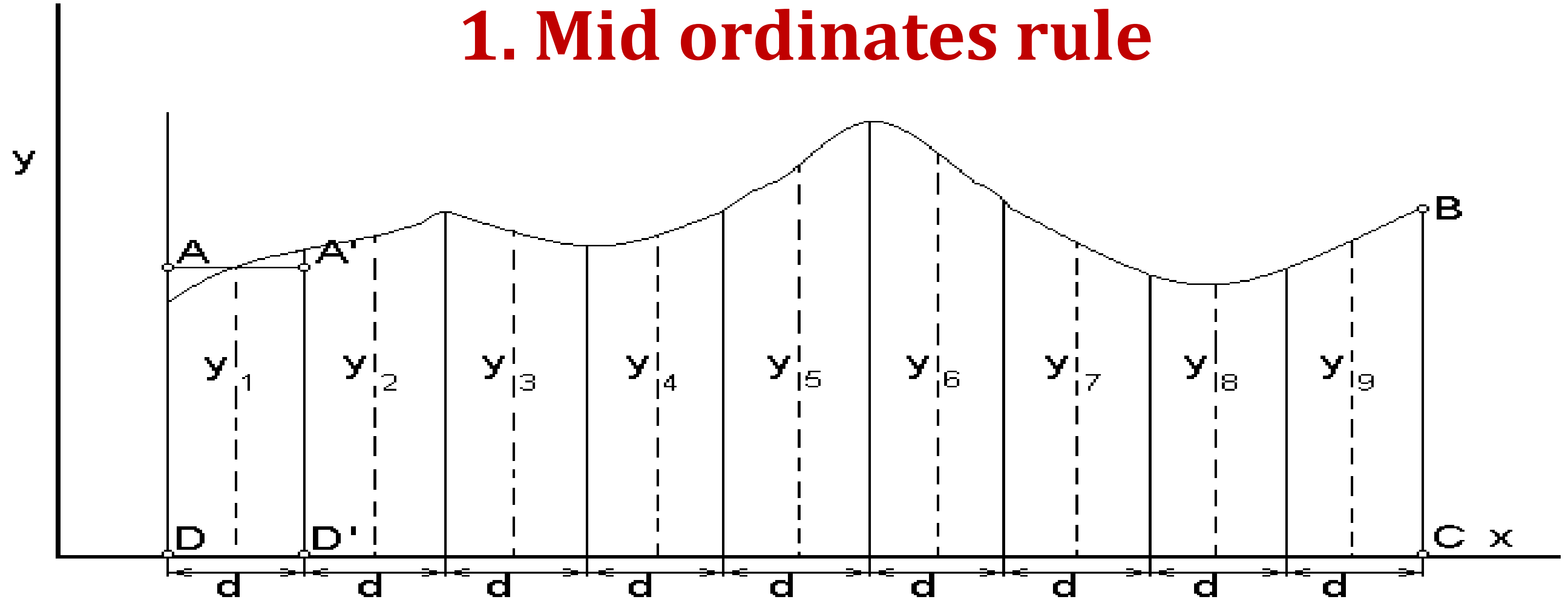
The choice of whether to use the appropriate Equation will depend on the triangle parts that are most conveniently determined; a decision ordinarily dictated by the nature of the area and the type of equipment available.

AREA BY OFFSETS FROM STRAIGHT LINES

- **Irregular tracts can be reduced to a series of trapezoids by observing right-angle offsets from points along a reference line.**

- **The spacing between offsets may be either regular or irregular, depending on the conditions.**

a. Regularly spaced offsets
1. Mid ordinates rule



To find the area of ABCD of the Figure above, the base is divided into number of equal strips width d.

As with the trapezoidal rule, the greater the number of intervals used the more accurate the result.

If each strip assumed to be a rectangular (see AA'D'D in figure above) and area of it is equal to base multiplied by mid-ordinate y_i.

Hence, the approximate area of ABCD is equal to:

$$\text{Area} = y_1 d + y_2 d + y_3 d + \ldots + y_n d$$

where

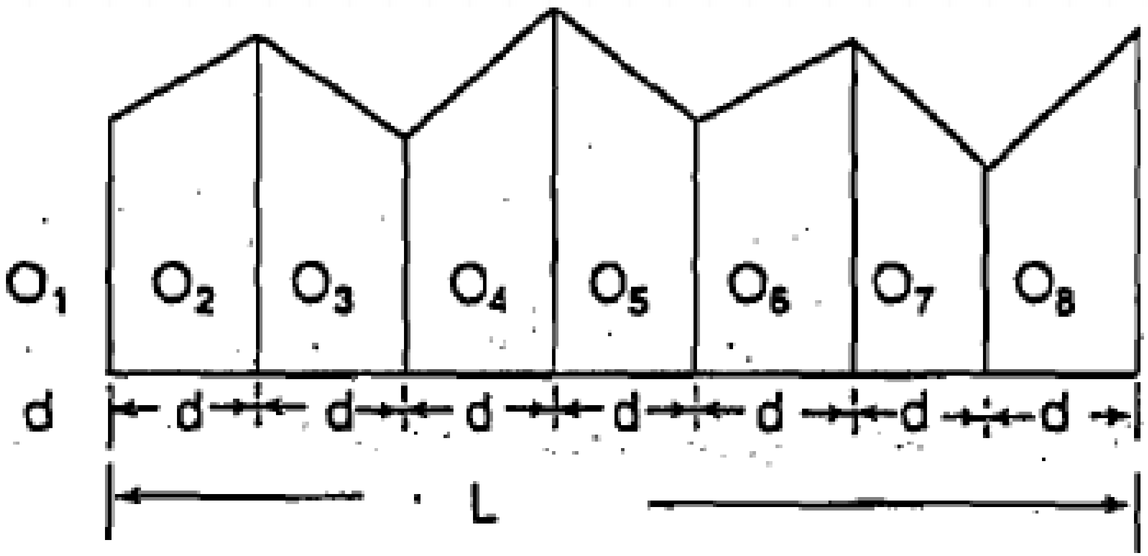
$$d = \frac{length\,of\ DC}{number\,of\ midordinates}$$

$$\text{Area} = d * [y_1 + y_2 + y_3 + \ldots + y_n]$$

Where,

n: is the number of strips

a. Regularly spaced offsets
2. Average ordinate rule.



If $O_1, O_2, \ldots, O_8$ are the ordinates to the boundary from the baseline

$$\text{Average ordinate} = \frac{O_1 + O_2 + O_3 + \ldots + O_7 + O_8}{8}$$

and

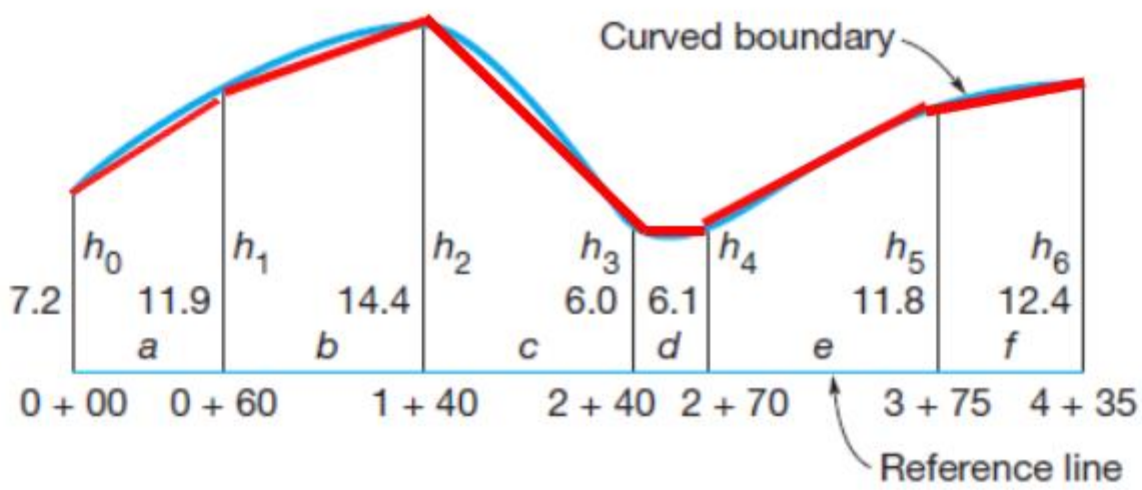
$$\text{area} = \text{average ordinate} \times \text{length}$$

$$= \frac{O_1 + O_2 + O_3 + \ldots + O_8}{8} \times L$$

b. Irregularly Spaced Offsets

- For irregularly curved boundaries like that in the Figure below, the spacing of offsets along the reference line varies.

- Spacing should be selected so that the curved boundary is accurately defined when adjacent offset points on it are connected by straight lines.



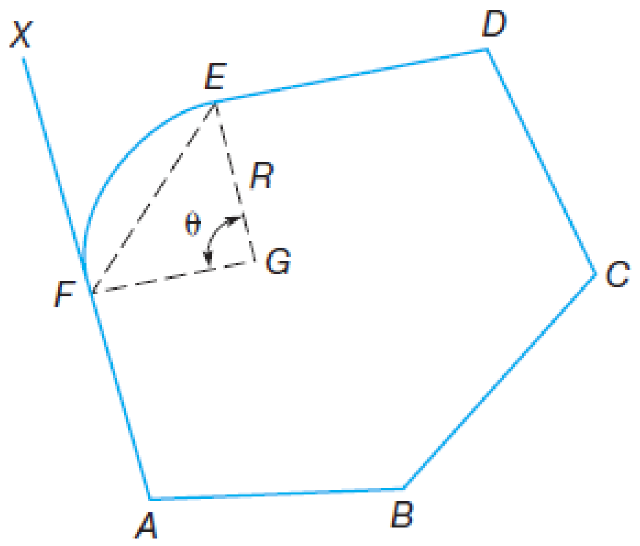
- **A formula for calculating area for this case is:**

$$\text{area} = \frac{1}{2}[a(h_0 + h_1) + b(h_1 + h_2) + c(h_2 + h_3) + \cdots]$$

where $a, b, c, \ldots$ are the varying offset spaces, and $h_0, h_1, h_2, \ldots$ are the observed offsets.

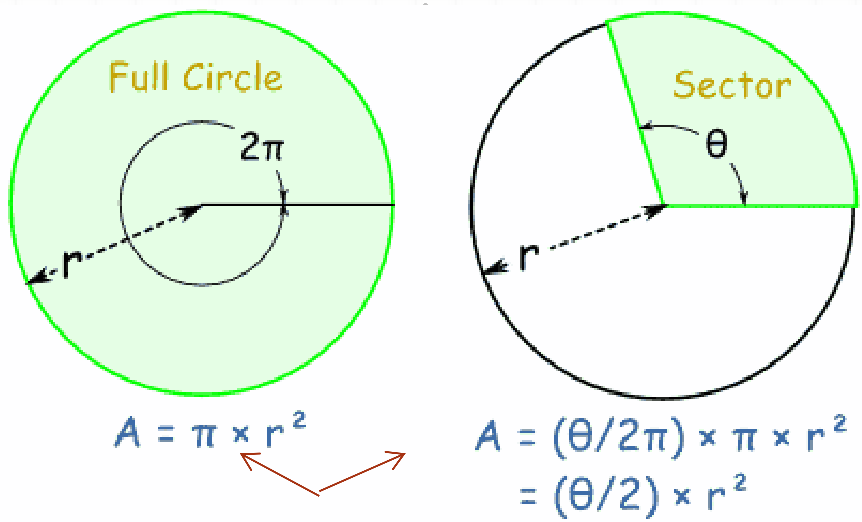
AREA OF PARCELS WITH CIRCULAR BOUNDARIES

- The area of a tract that has a circular curve for one boundary can be found by dividing it into two parts:



polygon **ABCDEGFA** and sector **EGF**.

To obtain the tract's total area, the **sector area is added to area ABCDEGFA** found by either the coordinate or DMD method.

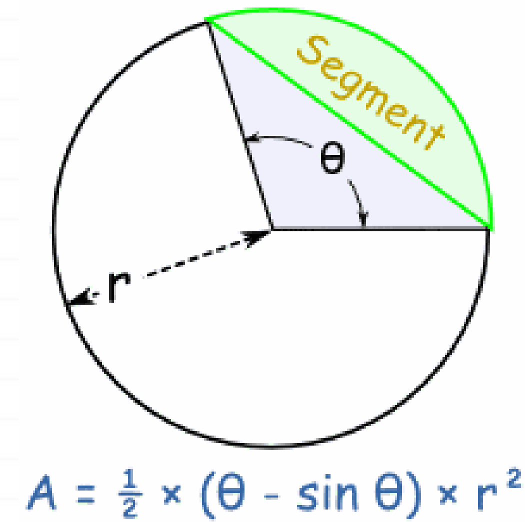


Remember:

Sin x Cos y=1/2[sin (x+y) + sin(x-y)]

we are using radians for the angles.

Area of Sector = ½ × θ × r^2 ***(when θ is in radians)***
Area of Sector = ½ × (θ × π/180) × r^2 ***(when θ is in degrees)***



The Area of a Segment is the area of a sector minus the triangular piece (shown in light blue here).

Area of Segment = ½ × (θ - sin θ) × r2 (when θ is in radians)

Area of Segment = ½ × ((θ × π/180) - sin θ) × r2 (when θ is in degrees)

Area Calculation

1- Double meridian distance DMD

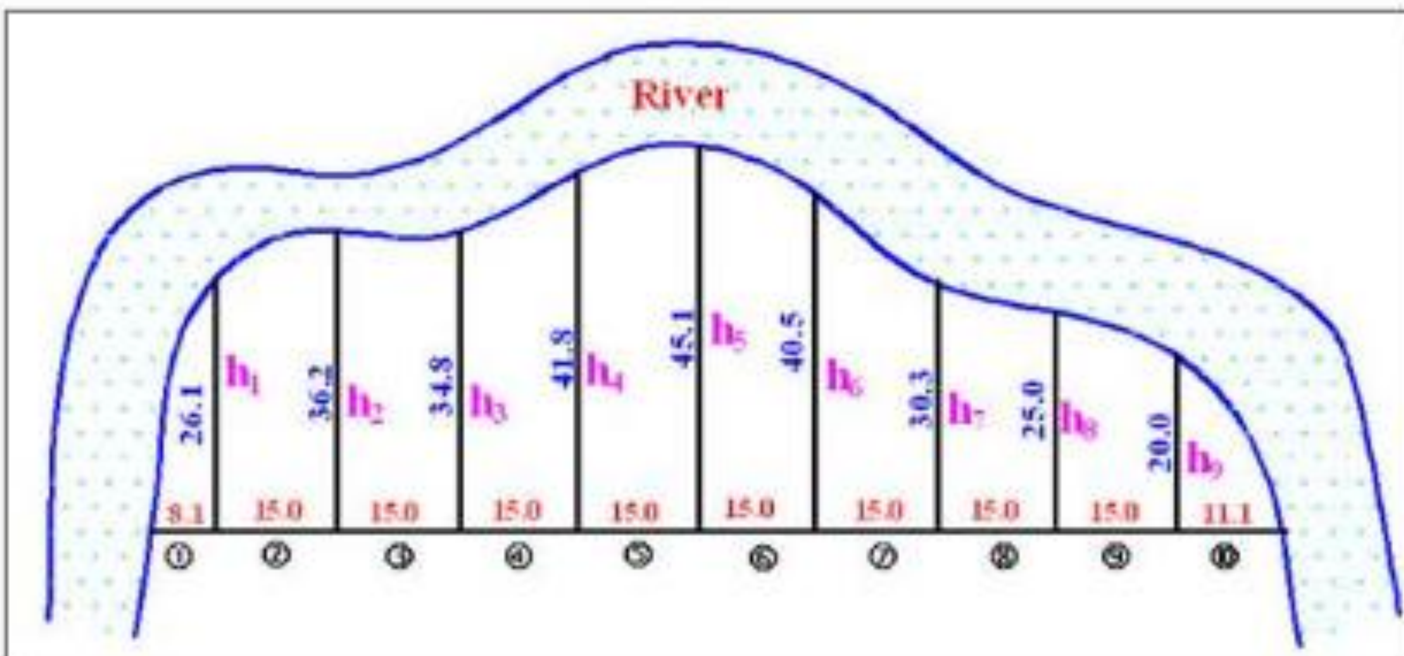
2- Coordinated method

3- Planimeter

- for irregular shapes
- graphical

Area Calculation/ Trapizoidal

- Semi irregular shapes
- Not very accurate
- Both end areas are taken as triangles
- Interior areas are considered trapezoids



A ① = (8.1 * 26.1) / 2 = 106 m^2

A Š = (11.1 * 20.0) / 2 = 111 m^2

Other areas are taken as trapezoids

$$Area = X\left(\frac{h_1 + h_n}{2} + h_2 + h_3 + \ldots\ldots + h_{n-1}\right)$$

where,

X = common interval between the lines

h = Offset measurement

n = number of offset measurements

$$Area = 15\left(\frac{26.1 + 20.0}{2} + 35.2 + 34.8 + 41.8 + 45.1 + 40.5\,30.3 + 25.0\right)$$

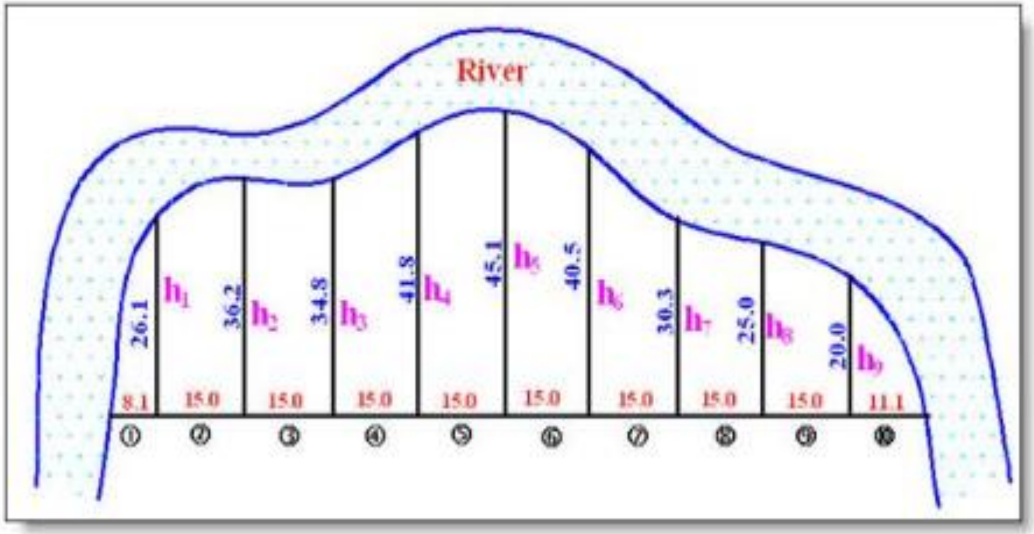
$= 4136$ m^2

Total Area = 4136 + 106 + 111 = 4353 m^2

Area Calculation/ Simpson's

$$A = \frac{interval}{3}(h_1 + h_n + 2\sum hodd + 4\sum heven)$$

- For semi irregular shapes
- Assumes that an odd number of offsets is involved and that lines joining the ends of three successive offset lines are parabolic in configuration
- More accurate than Trapizoidal.
- If offsets are even then last area is calculated using trapezoidal technique



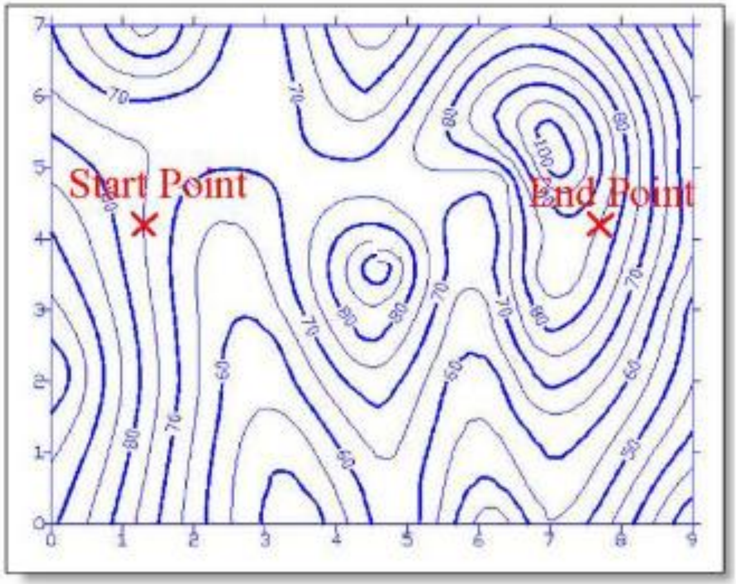
$$A = \frac{15}{3}[26.1 + 20.0 + 2(34.8 + 45.1 + 30.3) + 4(35.2 + 41.8 + 40.5 + 25.0)]$$

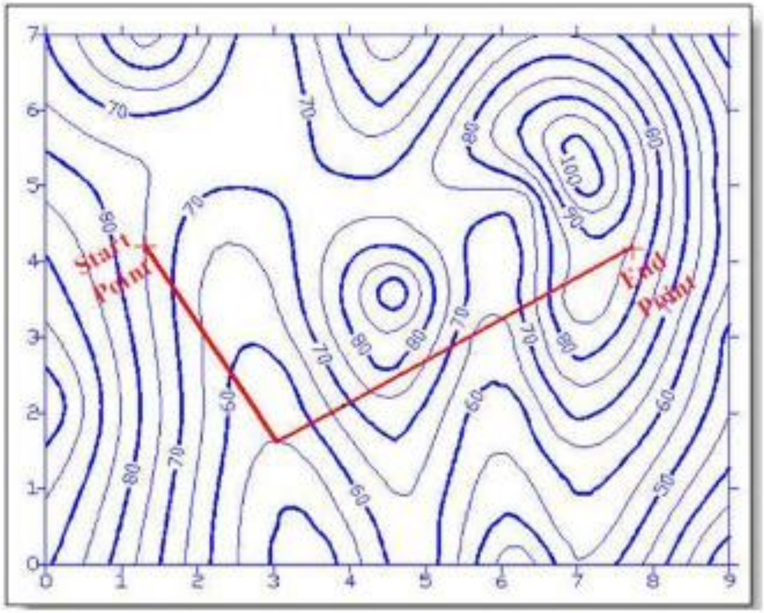
$$= 4183\ m^2$$

Total Area = 4183 + 106 + 111 = 4353 m^2

Complete Road Lay-Out Example

You are required to connect a road between the two marked points



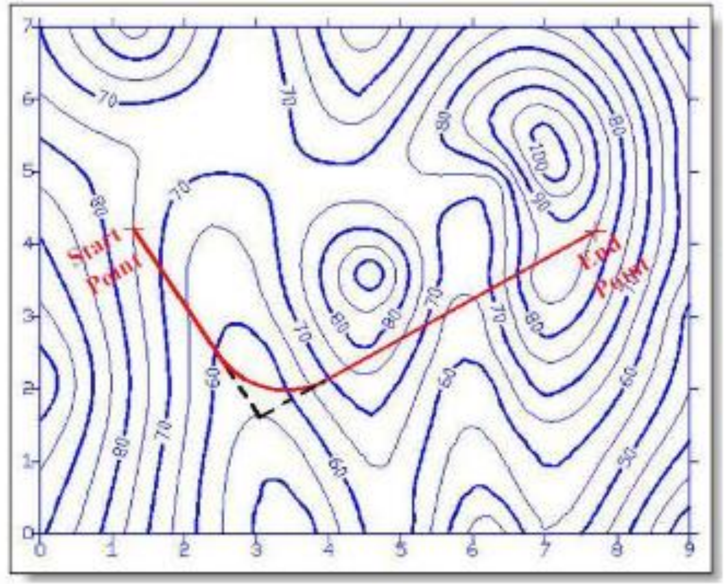


1. Connect the two points with straight sections. The best section is the direct connection. Due to large differences in elevations or presence of obstacles, the connecting road might include some turns.

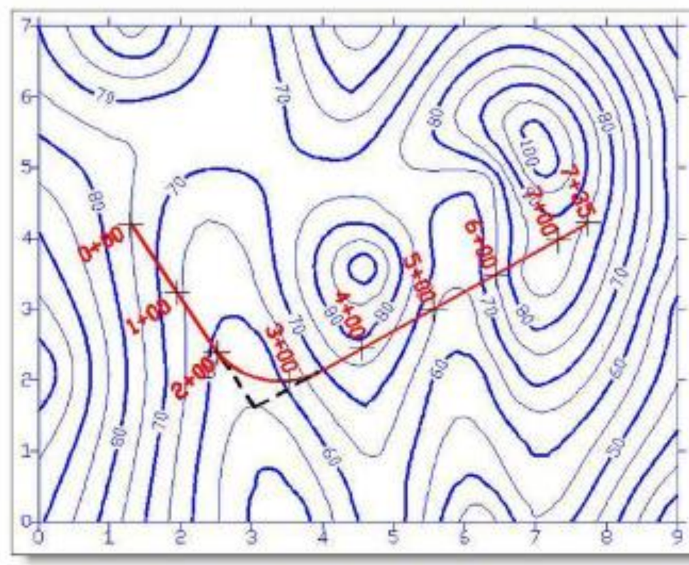
Shown lines are the plan of the center- line of the road.

Complete Road Lay-Out Example

2. **Design the turns. These turns are <u>circular horizontal</u> curves. The minimum radius of these curves is controlled by allowable super-elevation and design speed.**



3. **Locate the Stations.**

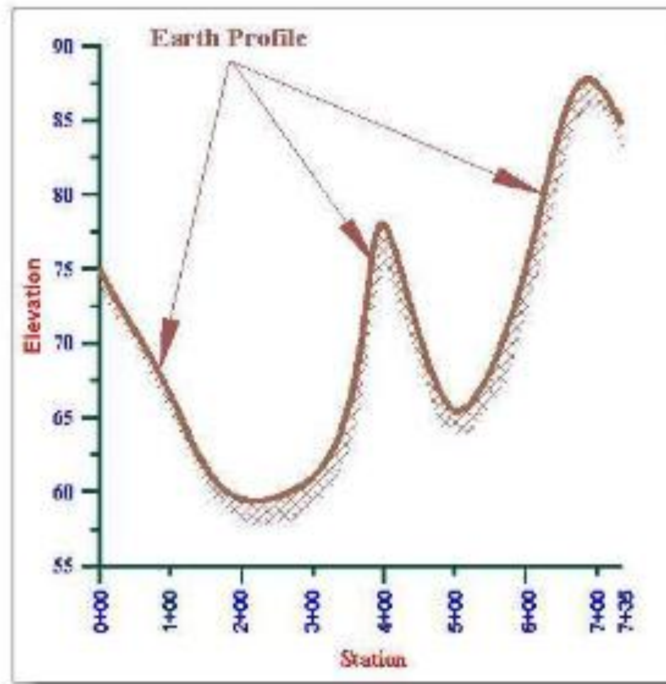


Complete Road Lay-Out Example

4. **Find the elevations of all the stations by reading them directly from the control map.**

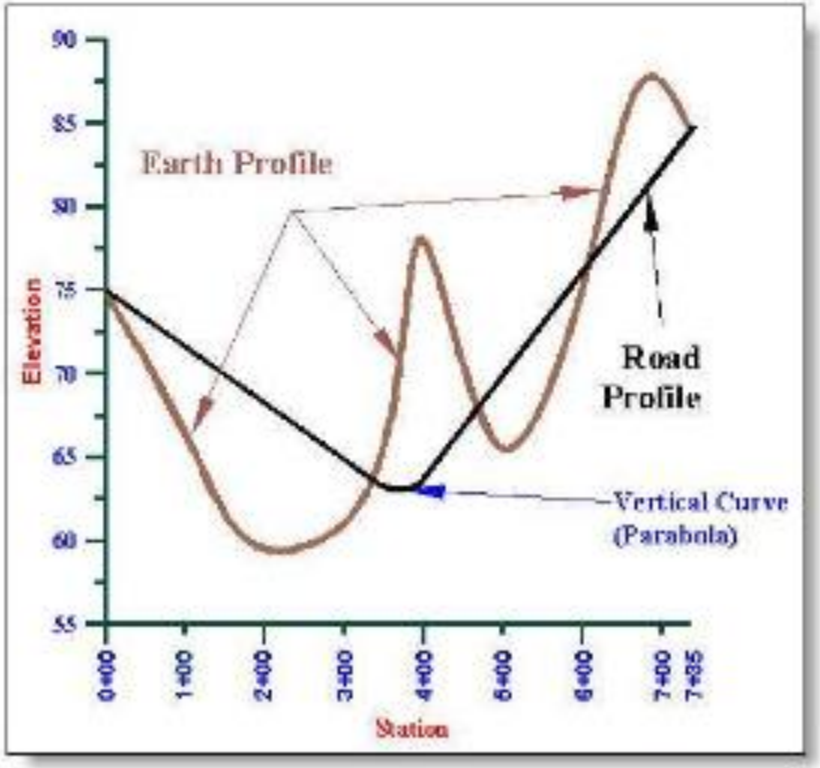
Station	Elevation
0 + 00	75
1 + 00	66.5
2 + 00	59.5
3 + 00	61
4 + 00	78
5 + 00	65.5
6 + 00	75
7 + 00	87.5
7 + 35	84.7

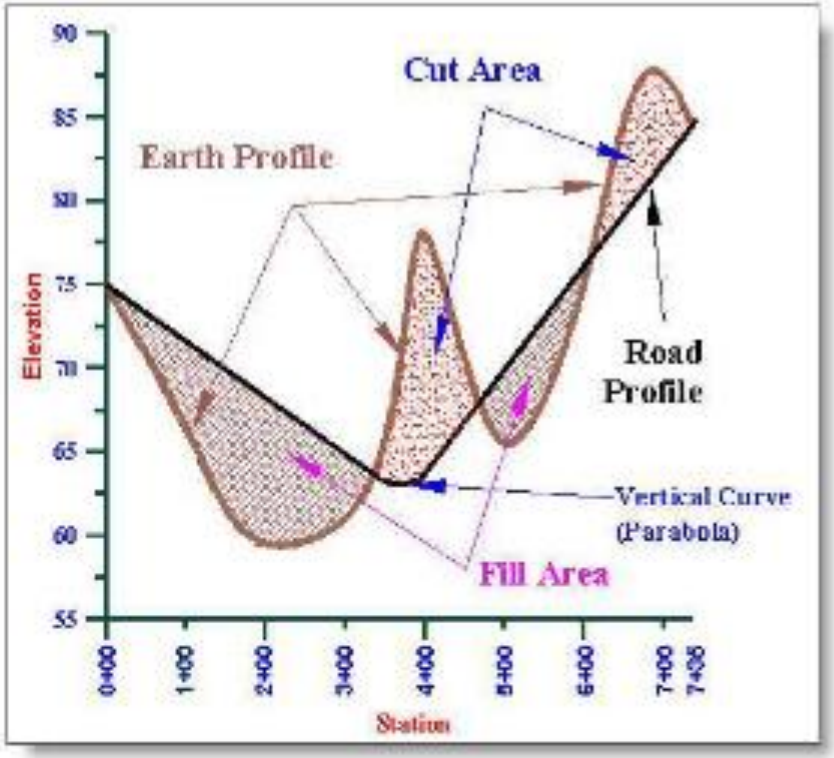
5. **From the obtained table, draw the earth profile of the center line of the road.**



Complete Road Lay-Out Example

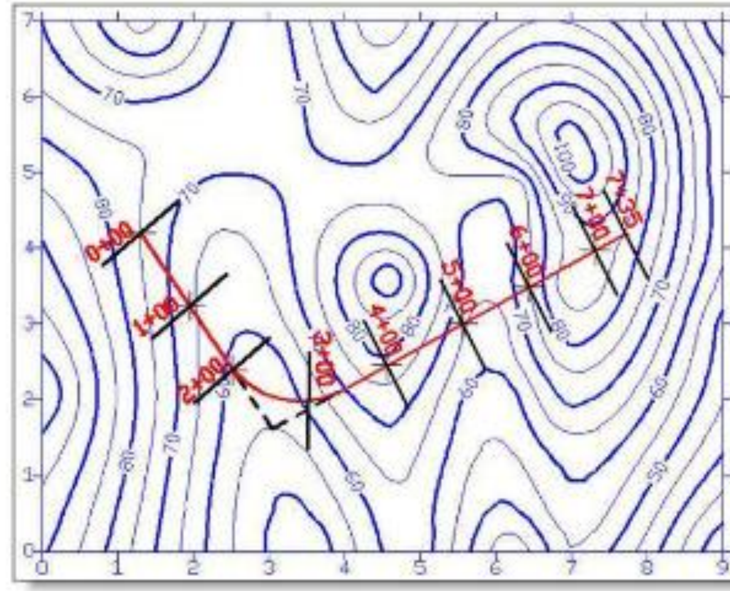
6. On the profile, connect the start and end points with straight lines. When connecting, select the connecting lines in a way to have the amount of cut to be equal to the fill amount. The slope of the lines is controlled by the maximum allowable vertical slope.



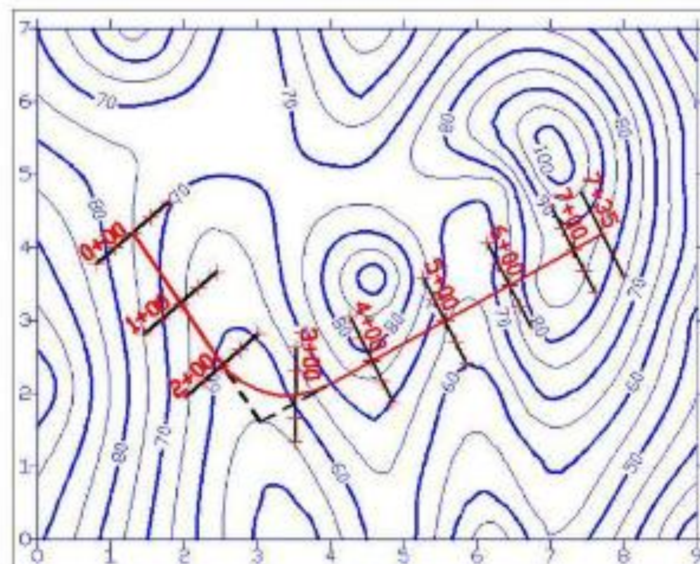


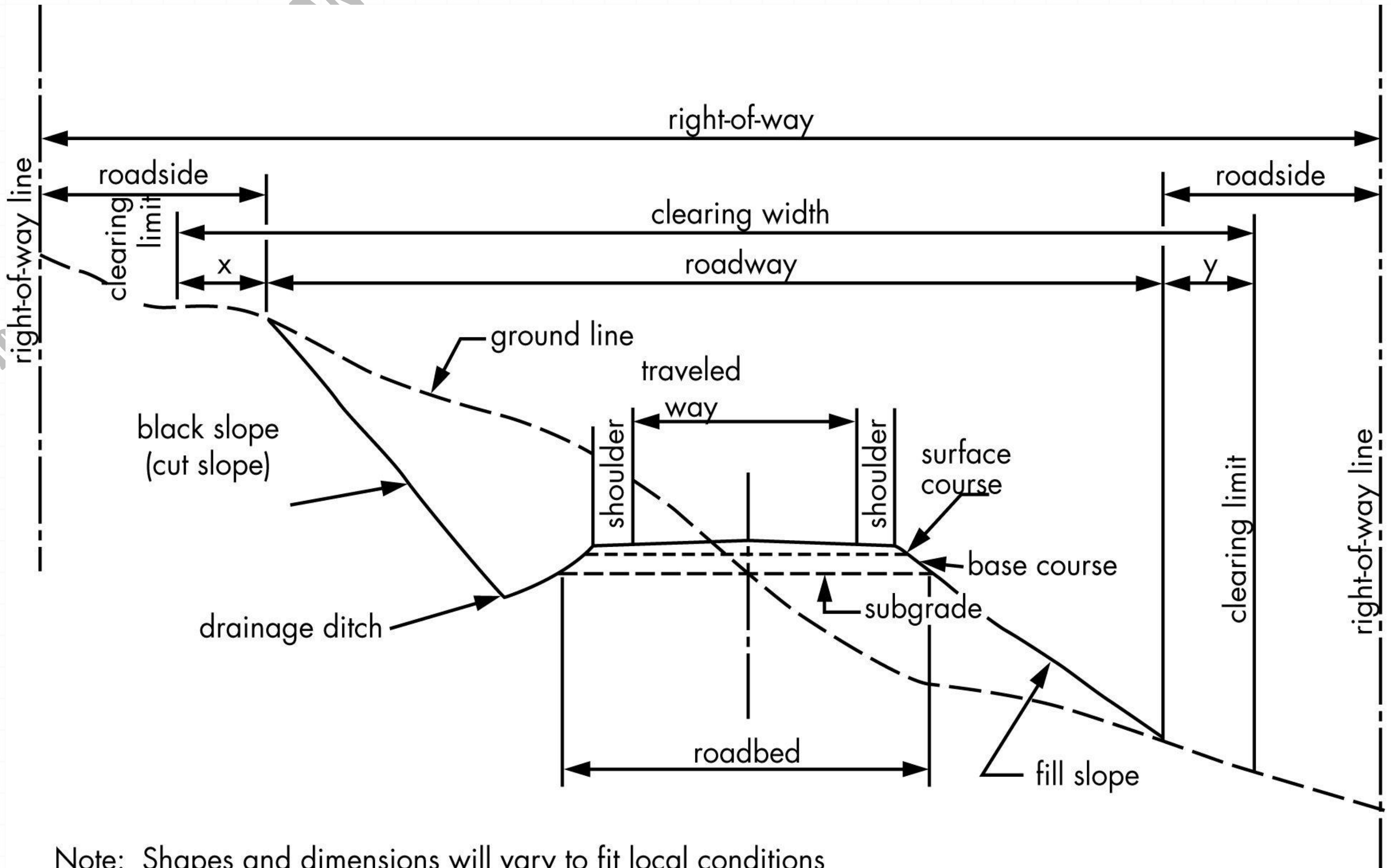
Complete Road Lay-Out Example

7. To get the cross section (ground elevations) at the different stations, perpendicular lines are drawn in the contour map on all the stations. The length of these lines is equal to the width of the right of way (ROW) of the road.



8. The constructed lines are divided into equal distances. Ground elevations are taken at these divisions.

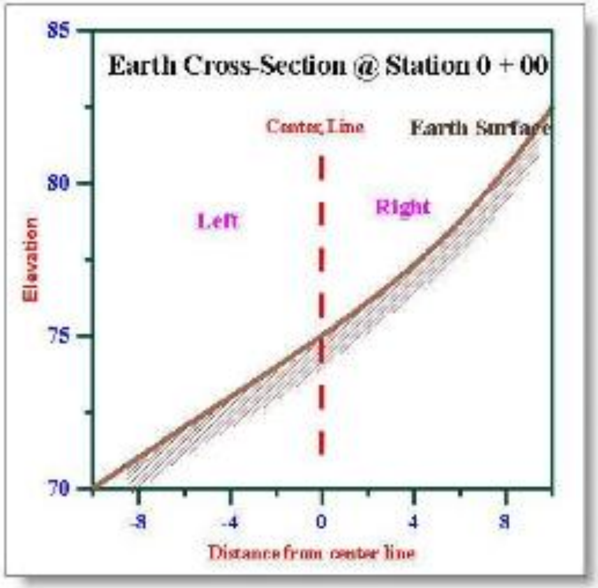




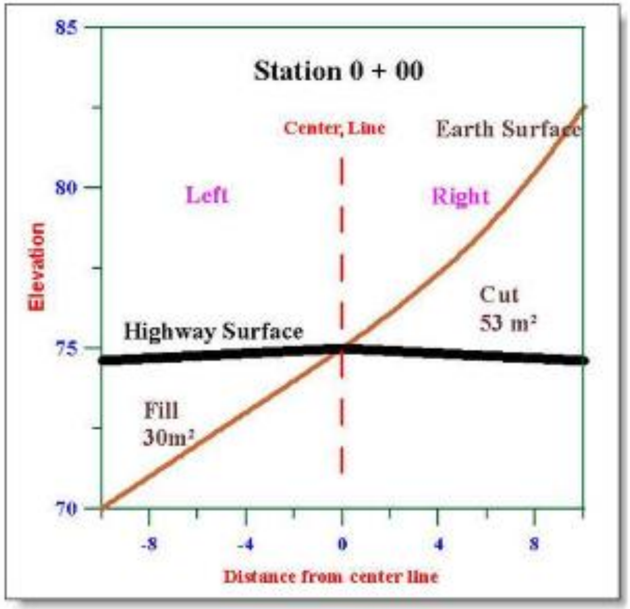
Note: Shapes and dimensions will vary to fit local conditions
See drawings for typical sections
x and y denote clearing outside of roadway

Complete Road Lay-Out Example

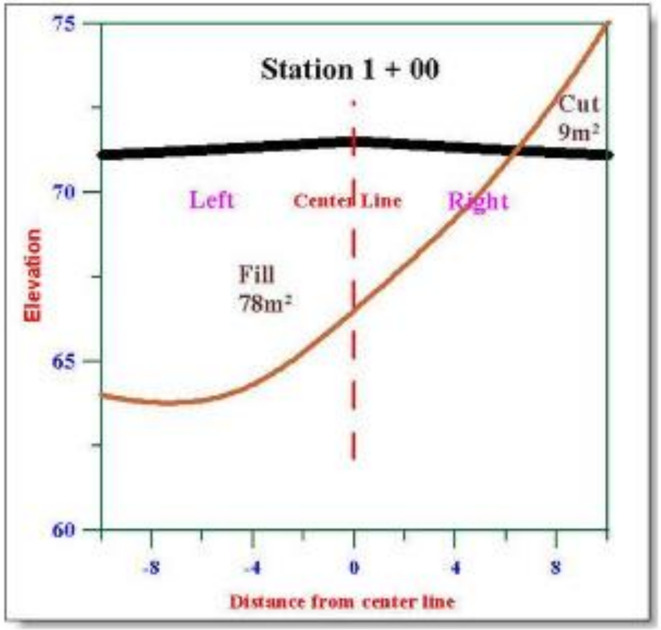
9. Obtained elevations at each station will be used to draw the cross sections.

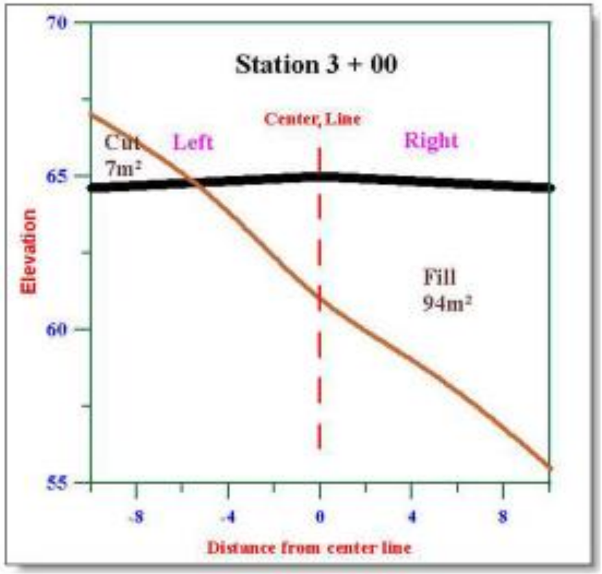


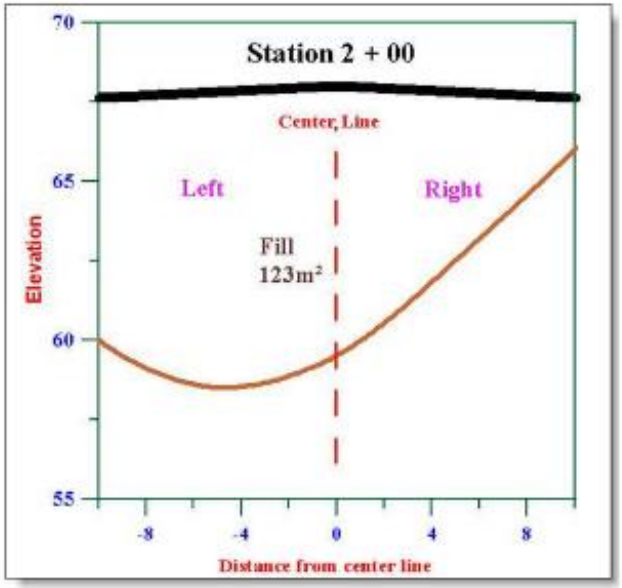
10. Get the elevation of the road center-line from the road profile. Then layout the pavement cross-section on the earth cross-section for each station.

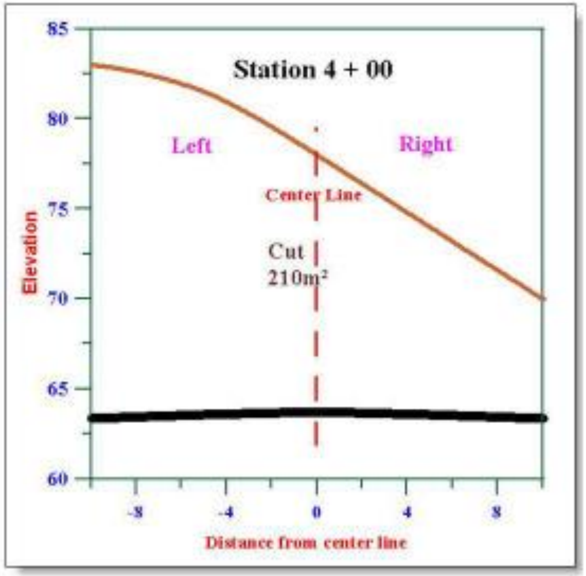


Complete Road Lay-Out Example

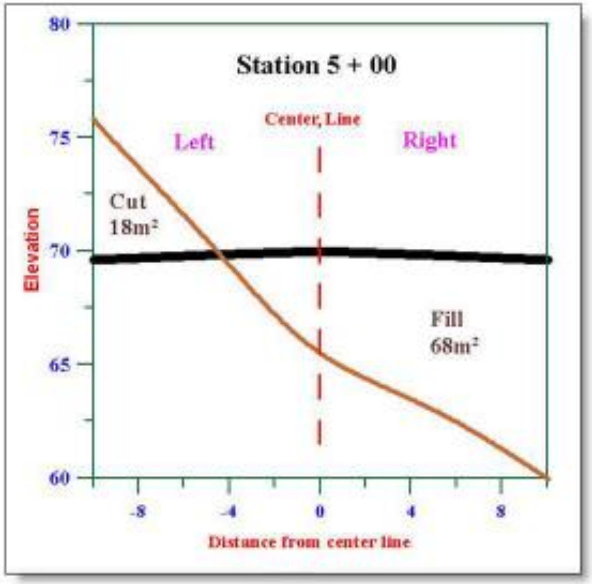


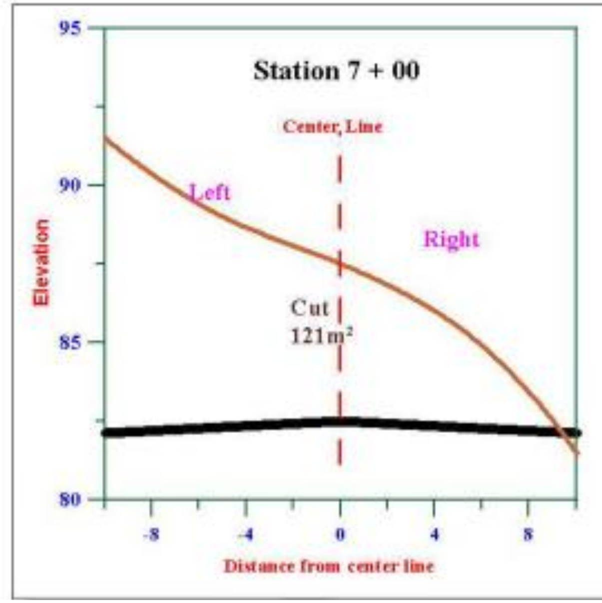


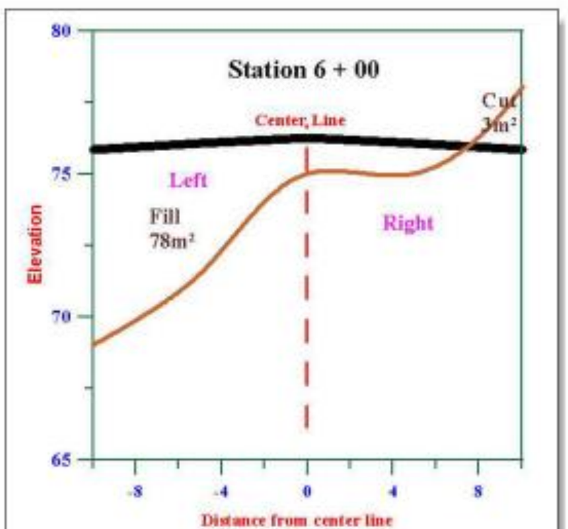


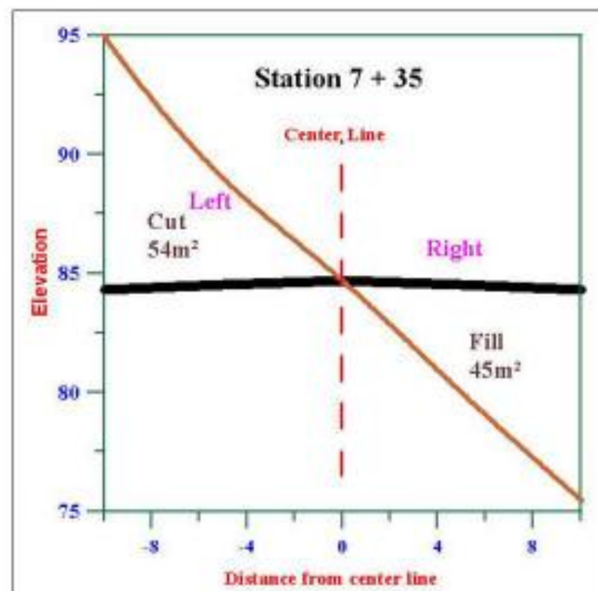


Complete Road Lay-Out Example









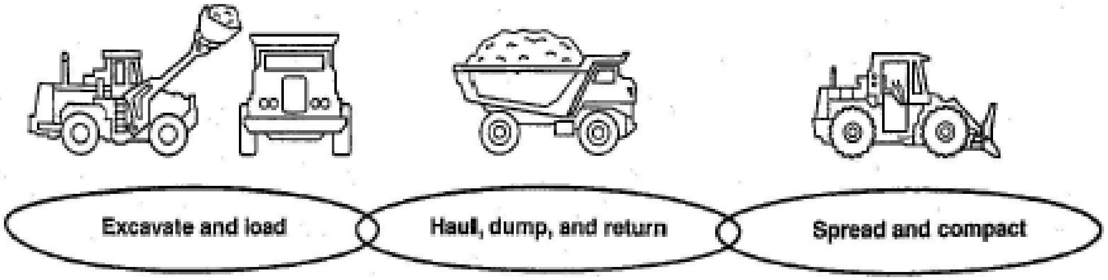
- In highway and railroad construction, excavation or cut material is used to build embankments or fill sections.
- Unless there are other controlling factors, a well-designed grade line should nearly balance total cut volume against total fill volume.
- To accomplish a balance, either fill volumes must be expanded or cut volumes shrunk (Expansion of fill volumes is generally preferred).
- This is necessary because, except for rock cuts, embankments are compacted to a density greater than that of material excavated from its natural state, and to balance earthwork this must be considered.
- The rate of expansion depends on the type of material and can never be estimated exactly.

- **However, samples and records of past projects in the immediate area are helpful in assigning reasonable factors.**

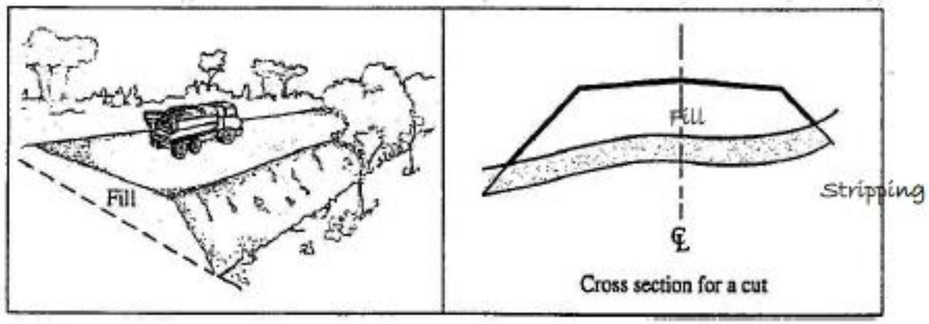
- **To investigate whether or not an earthwork balance is achieved, *cumulative volumes* are computed.**

- **This involves adding cut and expanded fill volumes algebraically from project beginning to end, with cuts considered positive and fills negative.**

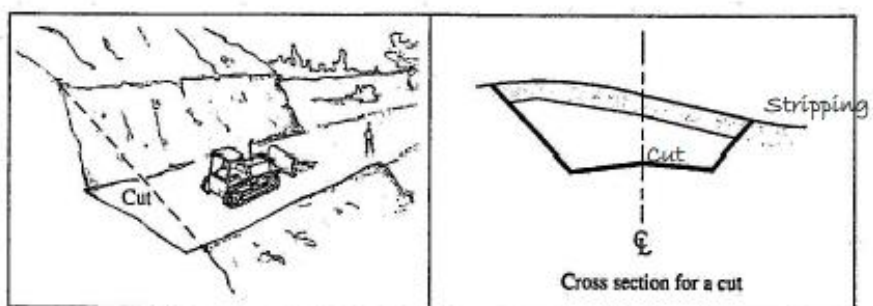
4. DISTRIBUTION OF THE ERATHWORK QUANTITIES.

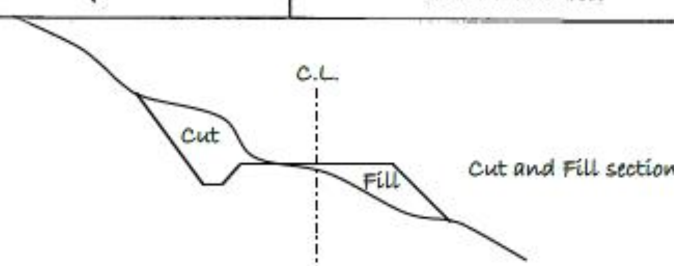


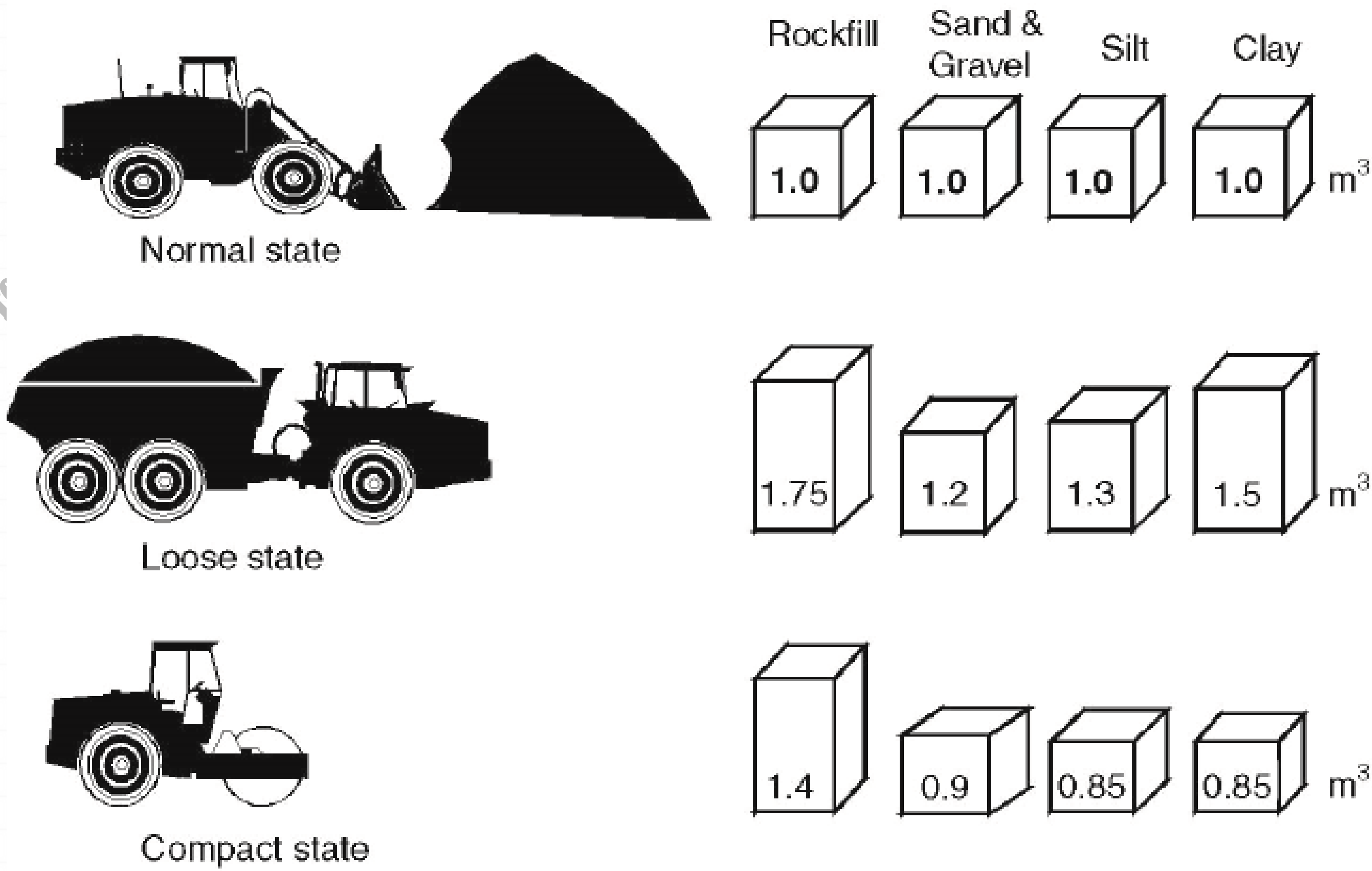
The (-ve) sign indicates an embankment.



The (+ve) sign indicates an excavation.





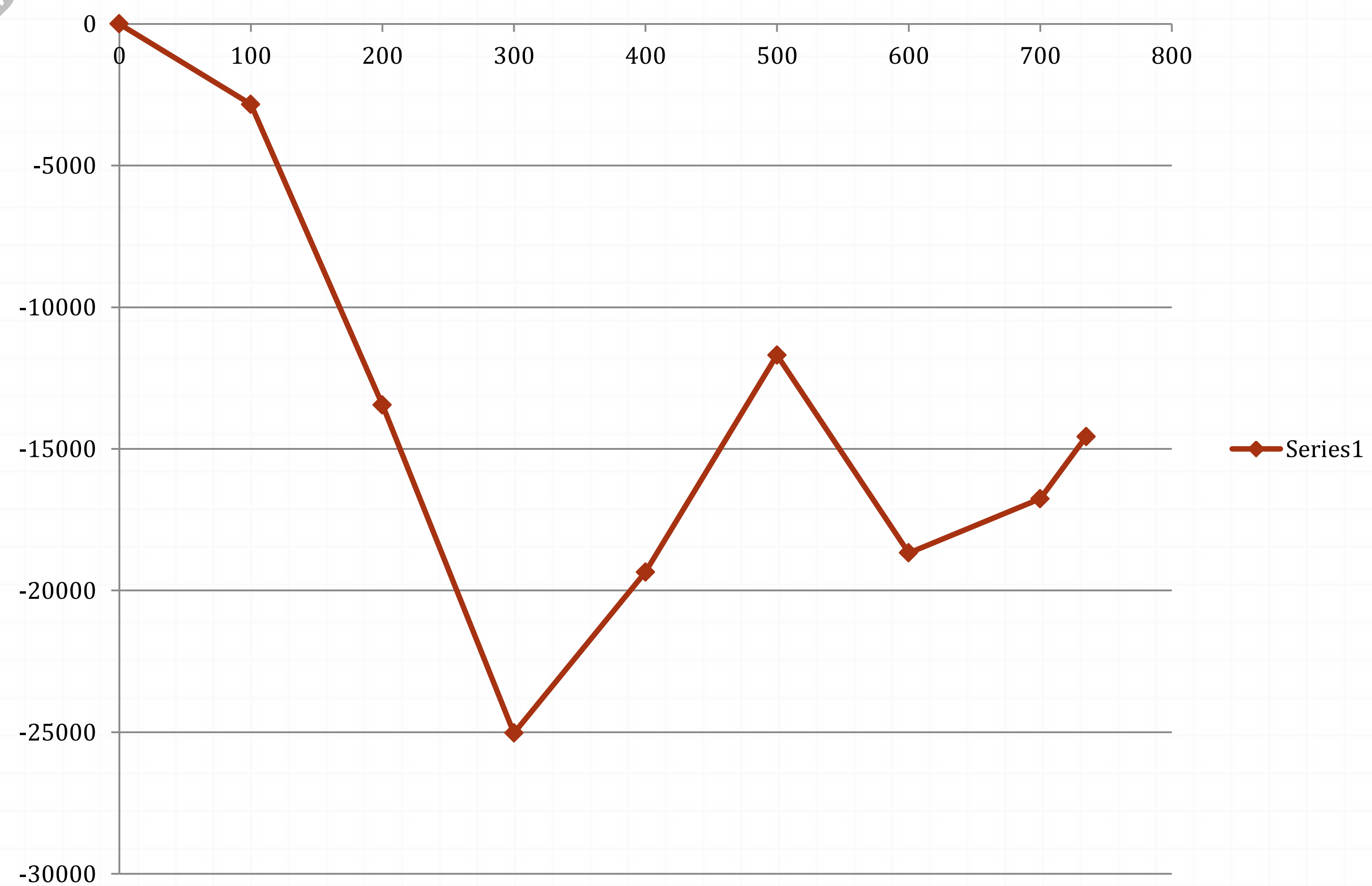

Rockfill
Sand & Gravel
Silt
Clay
1.0
1.0
1.0
1.0
m3
Normal state
1.75
1.2
1.3
1.5
m3
Loose state
1.4
0.9
0.85
0.85
m3
Compact state

Complete Road Lay-Out Example

11. Calculate the cut and fill areas for each cross section. Then, fill the mass diagram table.

Station	Distance	Cut Area (m2)	Fill Area (m2)	Cut Volume (m3)	Fill Volume (m3)	Shrinkage (10%) (m3)	Total Fill (m3)	Net Volume (m3)	Cumulative (m3)
									0
0+00		53	30						
	100			3100	-5400	-540	-5940	-2840	-2840
1+00		9	78						
	100			450	-10050	-1005	-11055	-10605	-13445
2+00		0	123						
	100			350	-10850	-1085	-11935	-11585	-25030
3+00		7	94						
	100			10850	-4700	-470	-5170	5680	-19350
4+00		210	0						
	100			11400	-3400	-340	-3740	7660	-11690
5+00		18	68						
	100			1050	-7300	-730	-8030	-6980	-18670
6+00		3	78						
	100			6200	-3900	-390	-4290	1910	-16760
7+00		121	0						
	35			3062.5	-787.5	-78.75	-866.25	2196.25	-14563.75
7+35		54	45						

Mass Haul Diagram



Mass Haul Diagram

- **Basic Definition of Mass Diagram:** A graphical representation of the cumulative amount of earthwork moved along the centerline and distances over which the earth and materials are to be transported.

- **Characteristics of Mass Curve:**

1- Rising sections of the mass curve indicates areas where excavating exceeds fill, whereas falling sections indicate where fill exceeds excavation.

2- Steep slopes reflect heavy cuts & Fills, while flat slopes indicate areas fro small amount of earthwork.

3- The difference in ordinates between any two points indicate net excess of excavation over embankment or vise versa.

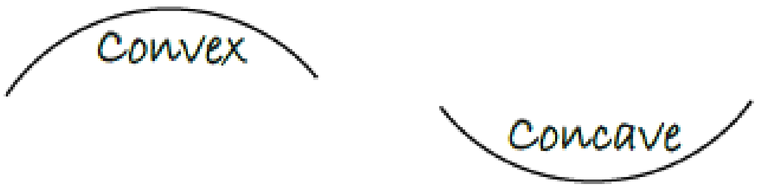
4- Any horizontal line drawn to intersect two points within the same curve indicates a balance of excavation (cut) and embankment (fill) quantities between the two points.

5- Points of zero slope represent points where roadway goes from cut to fill or from fill to cut.

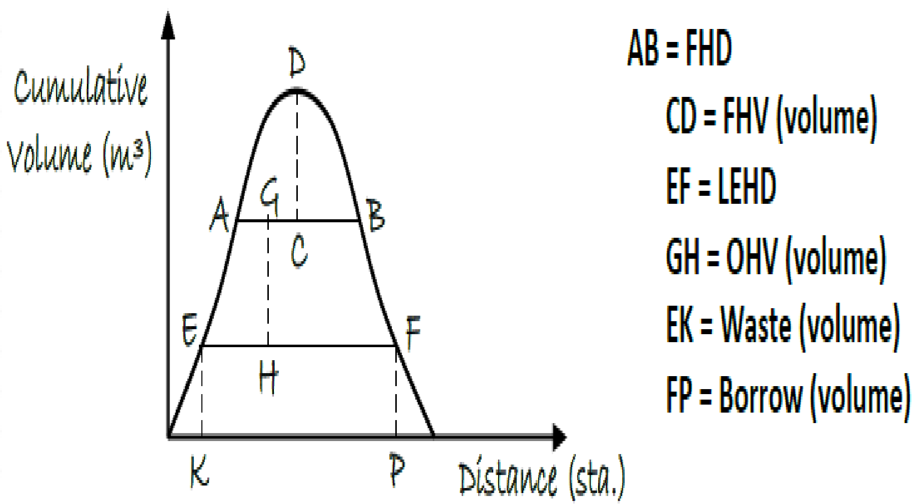
6- The highest or the lowest points of the mass haul diagram represents the crossing points between the grade line (roadway level) and natural ground level.

Mass Haul Diagram

7. The shapes of the mass-haul loops indicate the directions of haul. Thus, a convex loop shows that the haul from cut to fill is from left to right, whilst a concave loop indicates that the haul is from right to left.



8. The curve starts with zero-accumulated earthworks and the baseline is the zero balance line, i.e. when the curve intersects this line again the total cut and fill will balance. A line that is drawn parallel to the baseline so as to cut a loop is called a 'balancing line', and the two intersection points on the curve are called 'balancing points' as the volumes of cut and fill are balanced between them.



What does a Mass Diagram tell us?

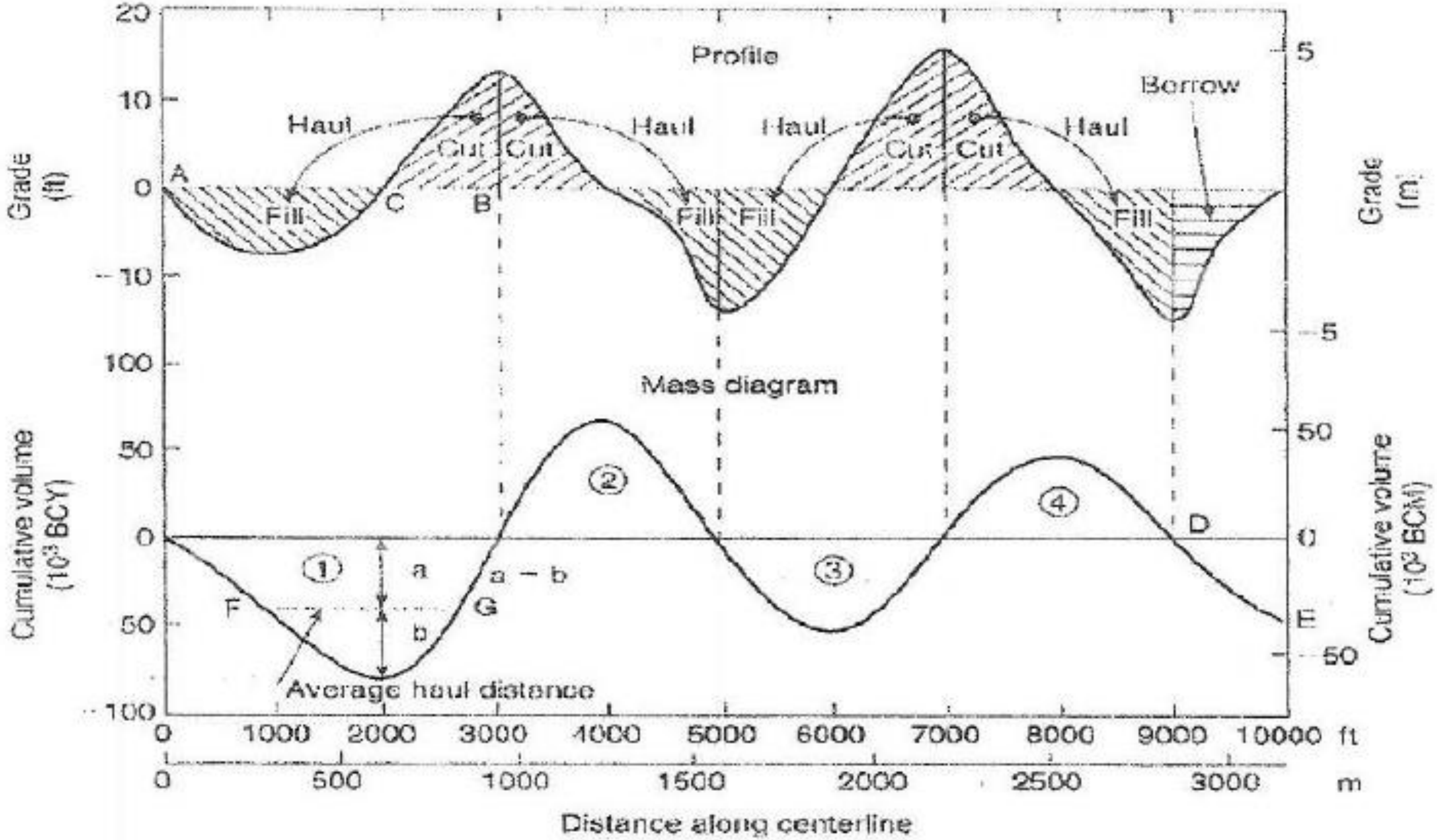
1.Mass diagrams determine the average haul, free haul, and overhaul on a given segment of roadway.

2.Mass diagrams tell the contractors and inspectors the quantity of material moved and how far it can be economically moved.

Definitions

- Haul : the transportation of excavated material from its original position to is final location in the work or other disposal area. This is also know as authorized haul.
- Average haul : determined from mass diagram. Average haul is the area of the mass diagram representing the number of cubic yard stations of haul between balance points divided by the ordinate of the mass which the yardage is hauled.
- Overhaul : the authorized hauling of excavation beyond the specified free-haul distance.
- Free haul : Average haul for project that is free.

Sample Mass Diagrams



The Mass-Haul Diagram:

The mass-Haul diagram is a graphical representation of the amount of earthworks involved in a highway schemes & the manner in which they may be most economically handled. It shows accumulated volume at any point along the center line.

(+) cut (excavation).

(-) fill (embankment).

Definitions:

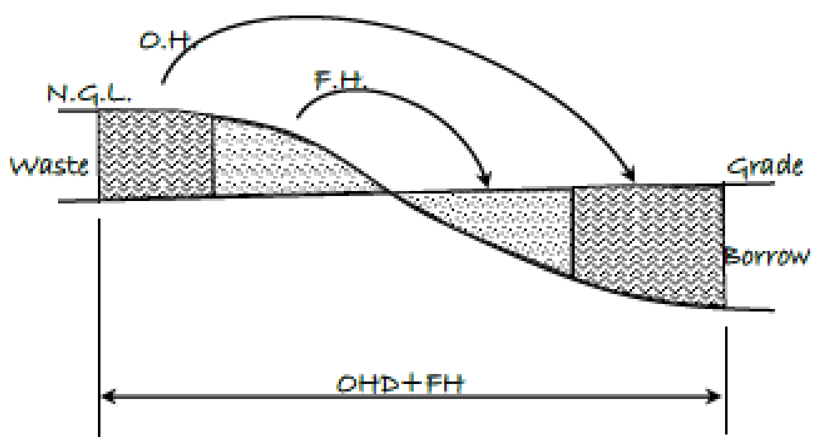
1. Haul: → Distance over which material is moved (m or station).

   → Volume-distance (m^3.m or m^3.sta.).

2. Free-Haul Distance (FHD): - The distance within which a contractor is paid a fixed amount per cubic meter of material irrespective of actual distance price, (free-haul charge=I.D./m^3).

3. Over-Haul Distance (OHD):- Distance beyond free-Haul for which extra charges are required for each (m^3. sta.), (over-haul charge=I.D./m3.sta.).

4. Limit of Economical Haul Distance (LEHD): - The maximum Over-Haul distance plus Free-Haul distance beyond which it is more economical to waste & borrow than to pay for over hauling.



Economic Haul Limit (LEHD) = FHD + L

$$L = \frac{C_B}{C_{OH}} \text{ sta.}$$

where:-

C_{OH} = cost of Over-Haul / m^3. sta.

C_B = cost of borrow material / m^3.

FHD = Free-Haul distance.

L = Maximum Over-Haul distance.

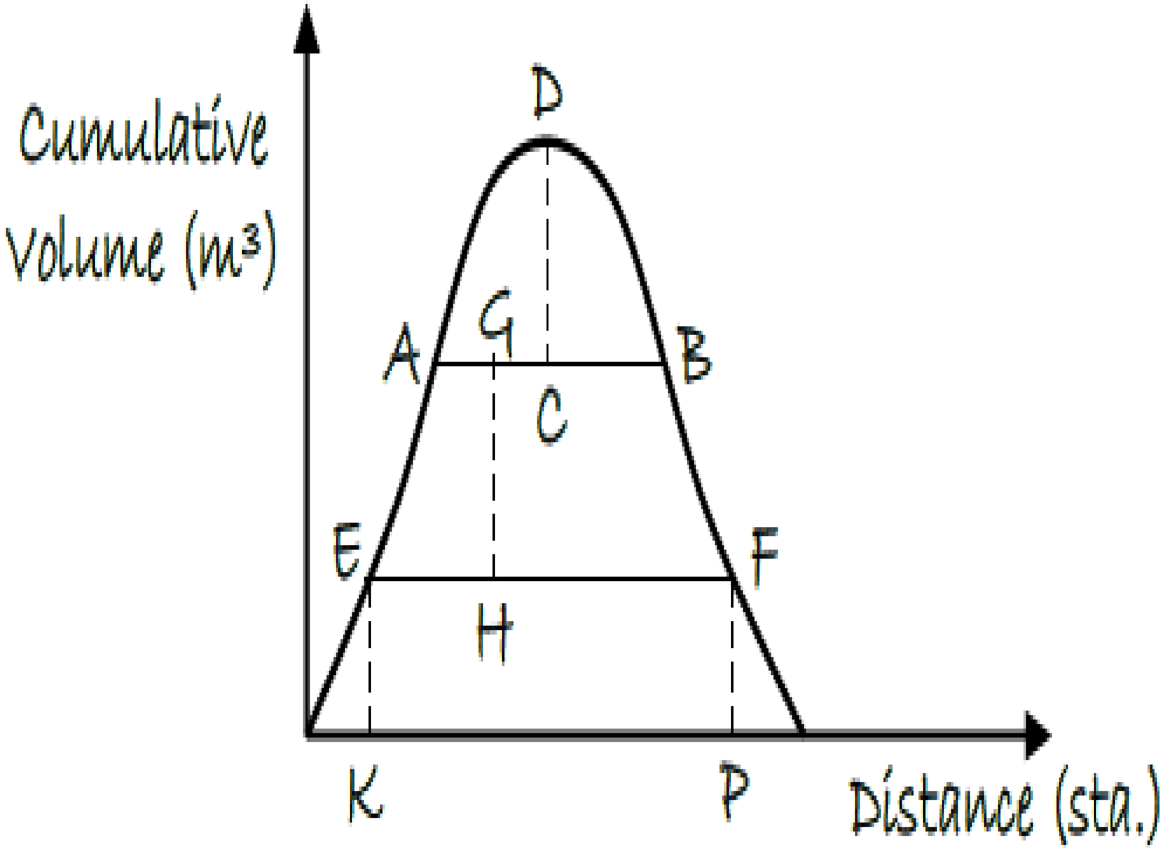
$$S.F. = \frac{V_f}{V_e}$$

Where: S.F = shrinkage factor.

V_f = volume of fill.

V_e = volume of excavation.

1. If the Subgrade soil is sand, silt, or clay then shrinkage (5-15) ≈ 10%.
2. If the Subgrade soil is rock, sand stone or lime stone then swell (bulking) (25-35) ≈ 30%.



AB = FHD

CD = FHV (volume)

EF = LEHD

GH = OHV (volume)

EK = Waste (volume)

FP = Borrow (volume)

To calculate overhaul volume, the median haul distance for material involved in overhaul is measured by drawing a horizontal line halfway between the free haul line and the balance line, as shown in the figure below.

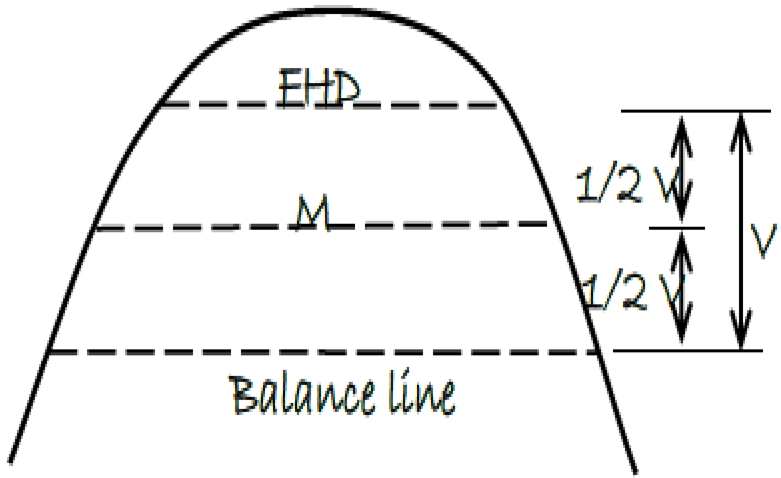
$$OHV = V(M - FHD)$$

where

V = the total volume involved in the overhaul.

M = median haul distance.

FHD = free-haul distance.

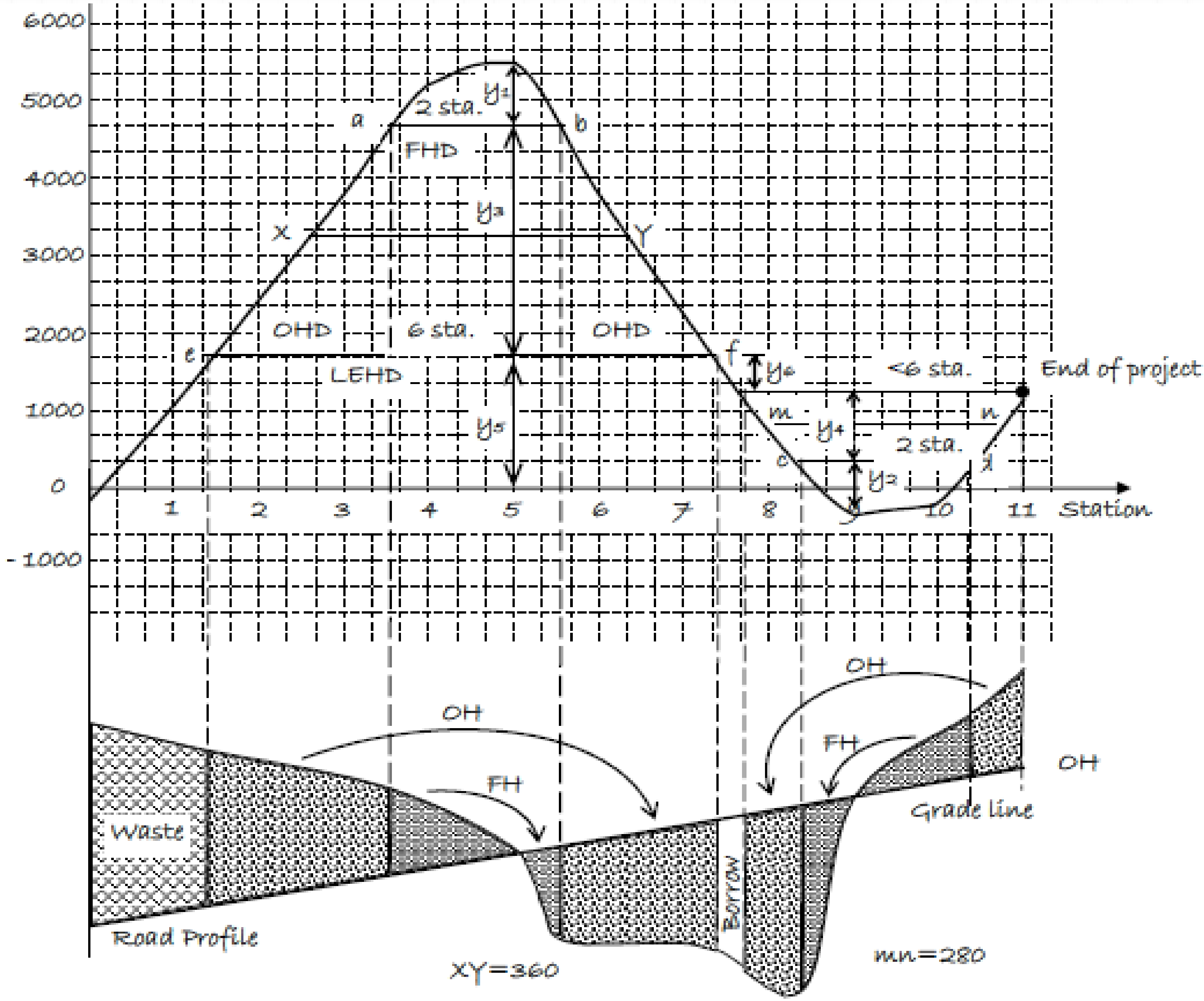


Example

Given the following end area for cut & fill. Complete the earthworks using shrinkage of 90% then prepare the M.H.D. & find the following:
a) Limit of economical haul.
b) Free-Haul volume.
c) Over-Haul volume.
d) Waste volume.
e) Borrowing volume.
f) Direction of hauling.
g) Total cost of the earthwork.

Giving that cost of Over-Haul = $30 /m3.sta.
Cost of Free-Haul = $70 /m3
Cost of borrow = $120 /m3
Free-Haul distance = 200 m.

Station	Area (m^2)		Volume (m^3)		Corrected Fill	Cumulated volume
	Cut	Fill	Cut(+)	Fill(-)		
0	10					0
			$\frac{10+12}{2}(100)=1100$			1100
1	12					
			1300			2400
2	14					
			1500			3900
3	16					
			1500			5400
4	14					
			$\frac{14+0}{2}(50)=350$	$\frac{10+0}{2}(50)=250$	275	5475
5		10				
				1300	1430	4045
6		16				
				1500	1650	2395
7		14				
				1300	1430	965
8		12				
				1000	1100	-135
9		8				
			300	200	220	-55
10	12					
			1400			1345
11	16					



6000
5000
4000
3000
2000
1000
0
- 1000
a
b
2 sta.
y_1
FHD
y_3
X
Y
OHD
6 sta.
OHD
e
f
LEHD
y_6
<6 sta.
End of project
m
n
y_4
y_5
2 sta.
c
d
y_2
1
2
3
4
5
6
7
8
9
10
11
Station
OH
OH
FH
FH
OH
Waste
Borrow
Grade line
Road Profile
XY=360
mn=280

Solution

- **Max overhaul distance = L= Cost of borrow/ Cost of over haul = 120/30 = 4 Stations = 400 m.**
- **Economic haul limit = L+FHD= 400+200 = 600 m = 6 Stations.**
- **b) Free-Haul volumes = y1+y2 = 700+900 = 1600 m3.**
- **c) Over-Haul volumes = y3+y4 = 3000+800 = 3800 m3.**
- **d) Waste volume = y5 = 1950 m3.**
- **e) Borrow volume = y6 = 600 m3.**
- **f) Total cost of the earthwork = y1*70+y3*70+y3*30*(XY-200)+y5*70+ y2*70+y4*70+y4*30*(mn- 200)+y6*120**
- **Cost of over-haul = y*cost of free-haul + y*cost of over-haul*distance**

Prof. TALEB AL-ROUSAN

Surveying

2104011365

Prof. Taleb M. Al-Rousan
Dept. of Civil Engineering
The Hashemite University

Prof. TALEB AL-ROUSAN

Chapter 9
GPS

GPS

- The Global Positioning System (GPS), originally Navstar is a satellite-based radionavigation system owned by the United States government and operated by the United States Space Force.

- It is one of the global navigation satellite systems (GNSS) that provides geolocation and time information to a GPS receiver anywhere on or near the Earth where there is an unobstructed line of sight to four or more GPS satellites.

- Obstacles such as mountains and buildings block the relatively weak GPS signals.

- Used to find the position of a point on earth (x, y, z) called Northing, Easting, and elevation.

GPS

- A minimum of four satellites must be tracked at any position to calculate the three unknowns(x, y, z) and to calculate the time difference between the satellite and receiver.
- Satellites weigh 860 Kg and span 5m, and include solar panels. They orbit the earth at 20,000 km in a period of 11 hrs and 58 min.
- The GPS consists of 24 satellites orbiting the earth in six orbits with additionally three spare satellites. Therefore, there are always at least four satellites visible at any point on earth.
- GPS satellites are Block II meaning a life span of seven years.
- Tracking stations are located at different positions on earth to send corrective data to the satellites.

GPS

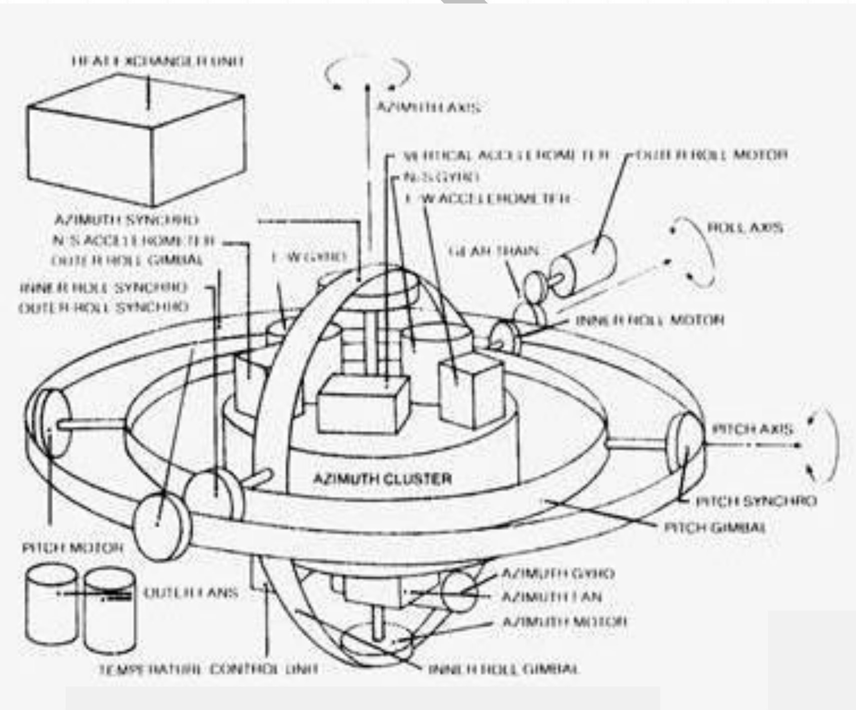
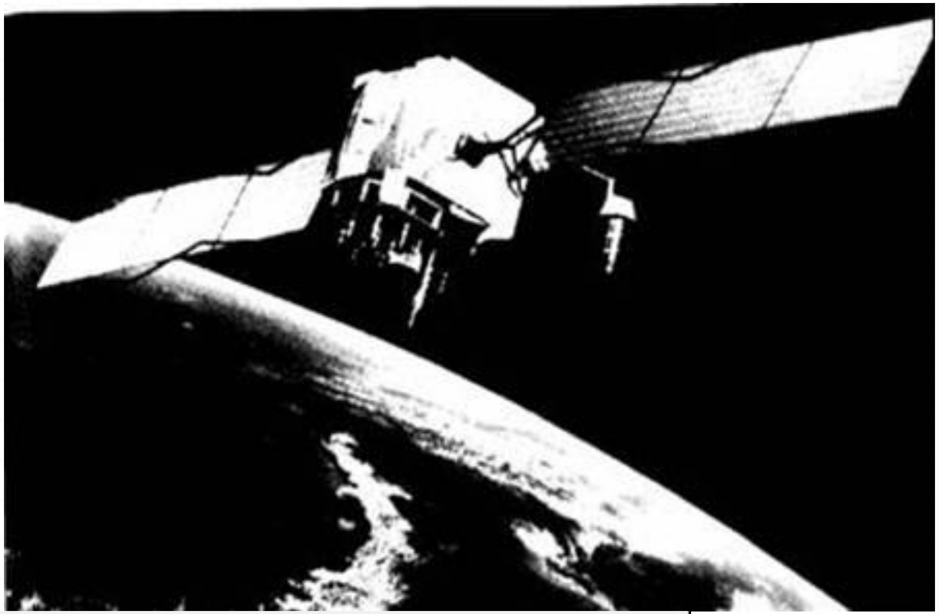
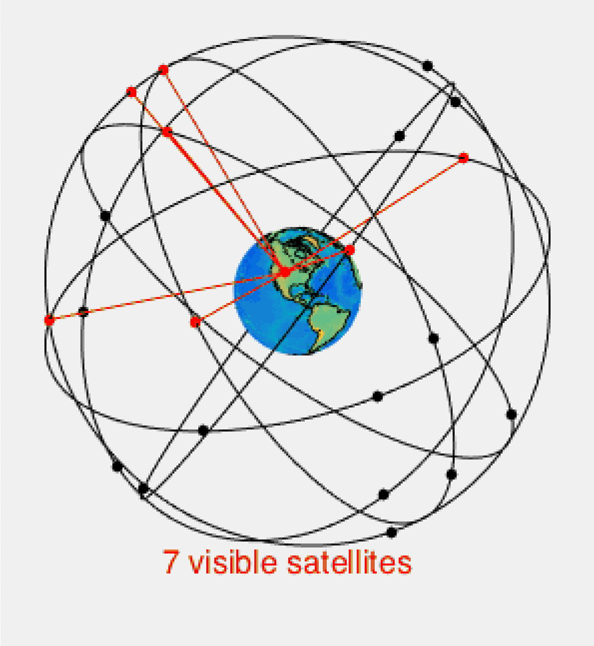
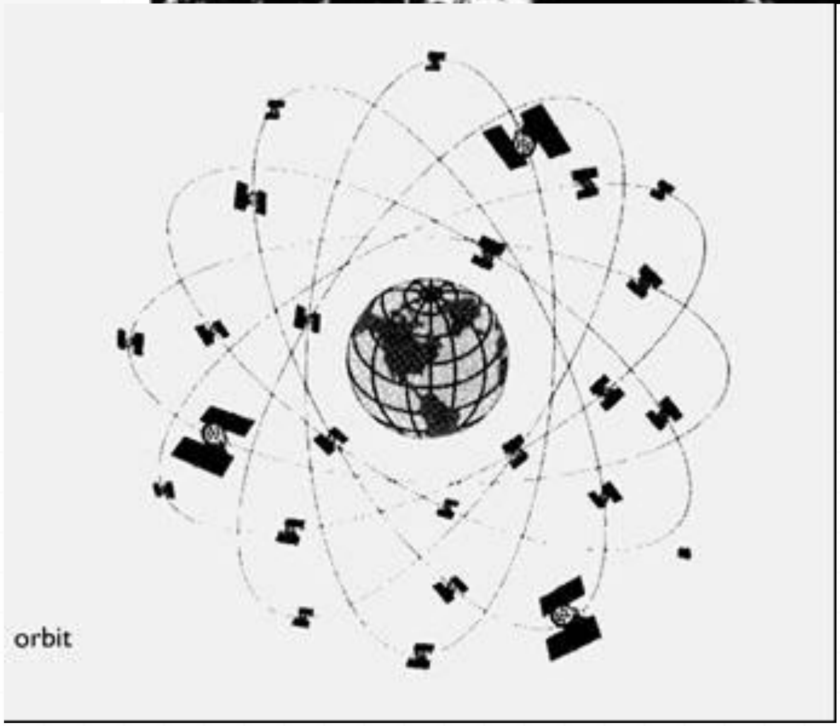
- **Russia has another 24 satellites (GLONASS) at an elevation of 19,100 km with an orbit time of 11 hrs15 min. GLONASS can be added to GPS devices, making more satellites available and enabling positions to be fixed more quickly and accurately, to within two meters .**

- **China's BeiDou Navigation Satellite System began global services in 2018, and finished its full deployment in 2020. There are also the European Union Galileo positioning system, and India's NavIC. Japan's Quasi-Zenith Satellite System (QZSS) is a GPS satellite-based augmentation system to enhance GPS's accuracy in Asia-Oceania, with satellite navigation independent of GPS scheduled for 2023**

- **The combination of GPS, GLONASS, and Galileo is called GNSS (Global Navigation Satellite System).**

GPS Accuracy

- When selective availability was lifted in 2000, GPS had about a five-meter (16 ft) accuracy. The latest stage of accuracy enhancement uses the L5 band and is now fully deployed. GPS receivers released in 2018 that use the L5 band can have much higher accuracy, pinpointing to within 30 centimeters or 11.8 inches.
- The US military's GPS is still more accurate than what most of us can access. It's good to the centimetre level, because it uses two frequencies to ping between satellites and receivers, one of which is encrypted.
- GPS-enabled smartphones are typically accurate to within a 4.9 m (16 ft.) radius under open sky . However, their accuracy worsens near buildings, bridges, and trees. High-end users boost GPS accuracy with dual-frequency receivers and/or augmentation systems


AZIMUTH AXIS
OUTER ROLL MOTOR
N-S GYRO
E-W ACCELEROMETER
AZIMUTH SYNCHRO
N-S ACCELEROMETER
OUTER ROLL GIMBAL
E-W GYRO
ROLL AXIS
INNER ROLL SYNCHRO
OUTER ROLL SYNCHRO
INNER ROLL MOTOR
PITCH AXIS
AZIMUTH CLUSTER
PITCH SYNCHRO
PITCH GIMBAL
PITCH MOTOR
AZIMUTH GYRO
AZIMUTH MOTOR
TEMPERATURE CONTROL UNIT
INNER ROLL GIMBAL


7 visible satellites

orbit

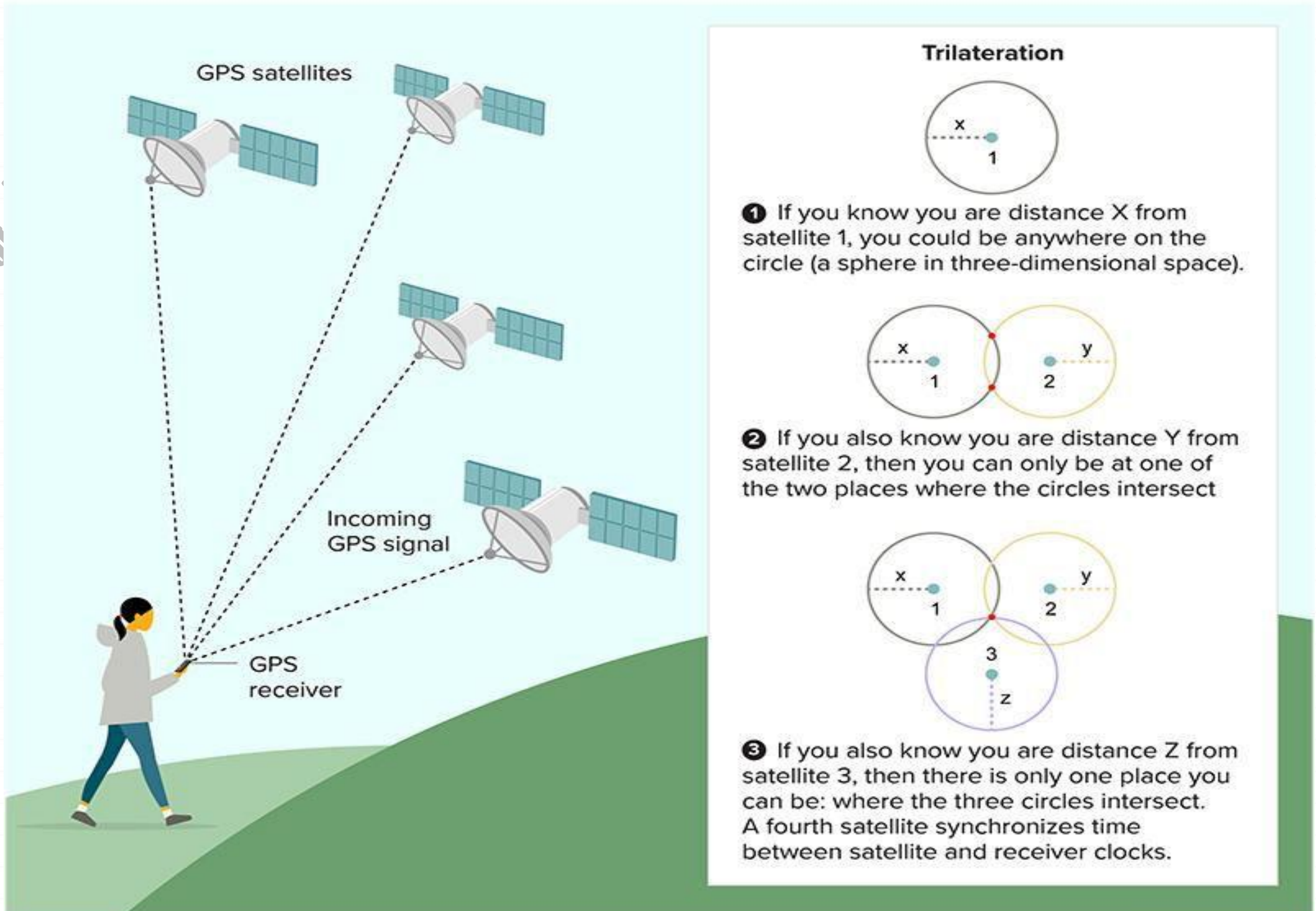
GPS Receivers

- A GPS Receiver is a L-band radio processor capable of solving the navigation equations in order to determine the user position, velocity and precise time (PVT), by processing the signal broadcasted by GPS satellites.

- Any navigation solution provided by a GNSS Receiver is based on the computation of its distance to a set of satellites, by means of extracting the propagation time of the incoming signals traveling through space at the speed of light, according to the satellite and receiver local clocks.

How GPS works

A GPS receiver, like the one in your smartphone, pinpoints its location on Earth's surface by analyzing its distance to three GPS satellites; a fourth satellite synchronizes clocks in the receiver and satellites.



SOURCE: UNAVCO

KNOWABLE MAGAZINE

GPS Receivers

Receivers can be categorized by their type in different ways, and under different criteria.

- Receivers can be stand-alone, or may benefit from corrections or measurements provided by augmentation system or by receivers in the vicinities (DGPS).
- Receivers might be generic all purpose receivers or can be built specifically having the application in mind: navigation, accurate positioning or timing, surveying, etc.
- In addition to position and velocity, GPS receivers also provide time. An important amount of economic activities, such wireless telephone, electrical power grids or financial networks rely on precision timing for synchronization and operational efficiency. GPS enables the users to determine the time with a high precision without needing to use expensive atomic clocks.

GPS Receivers

- **The initial purpose of the GPS system was military but with the free availability of GPS signals and the availability of cheap GNSS receivers, the GPS technology is having a pervasive use in civil, industrial, scientific areas.**
- **Currently the use of GPS in Civil Applications is generalized, and it is well known that GPS Receivers have been spread very fast as well as the manufacturers dedicated to this (e.g. CSR, BroadCom, Garmin,…).**

GPS Receivers

- **Range in ability and cost from high precision used in surveying operations, to mapping and Geographic Information System (GIS), to marine navigation receivers, and finally to positioning and low-precision mapping and GIS receivers.**

- **They differ in 1) Number of satellites that can be tracked at one time. 2) Whether receiver is double frequency or single frequency receiver (L1 and / or L2), in addition can it receive code phase or carrier phase.**

- **Higher cost receivers are dual frequency, require shorter observation times, and can be used for real-time positioning.**

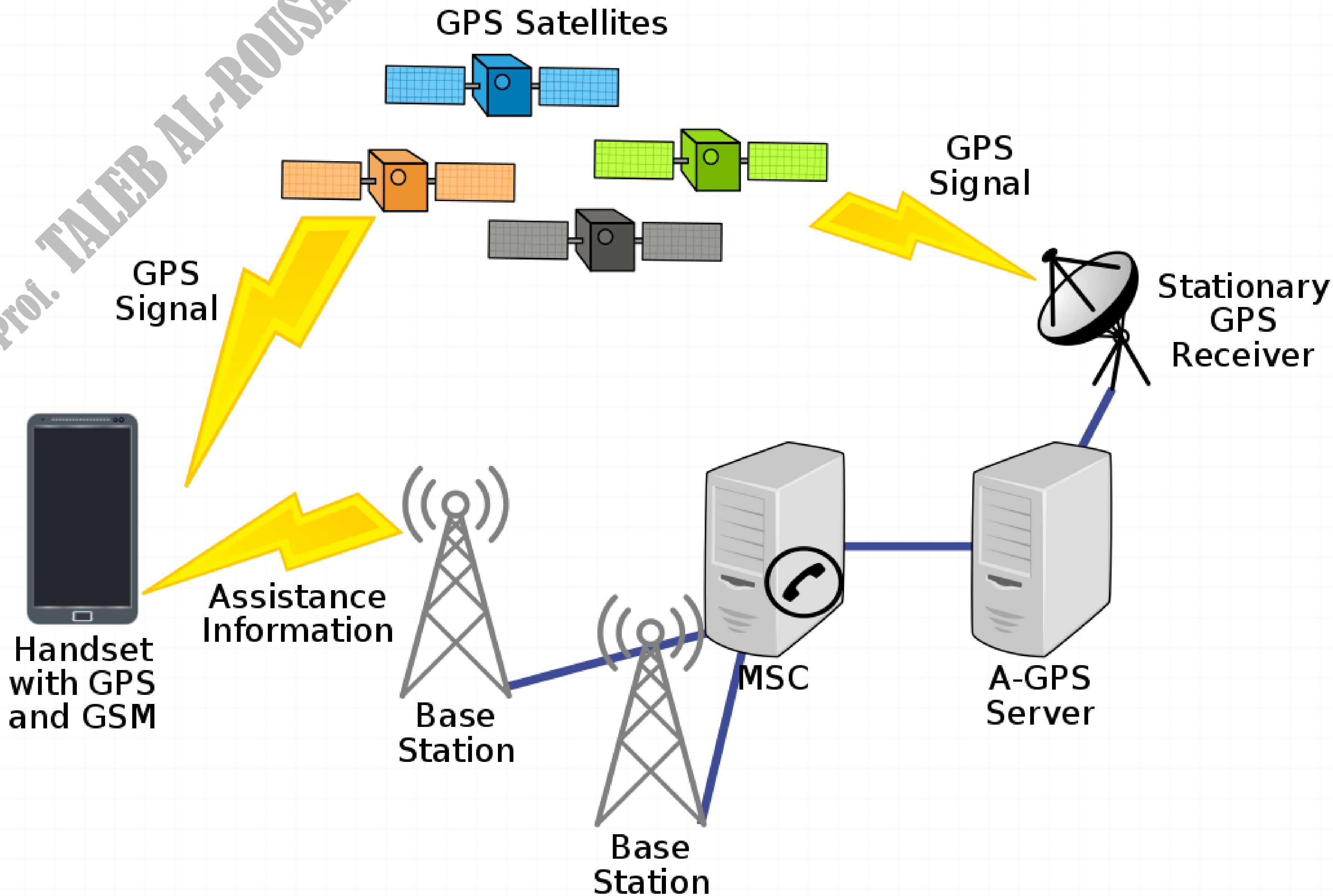
- **General purpose receivers track one satellite at a time. Sequencing from satellite to satellite as tracking progresses.**

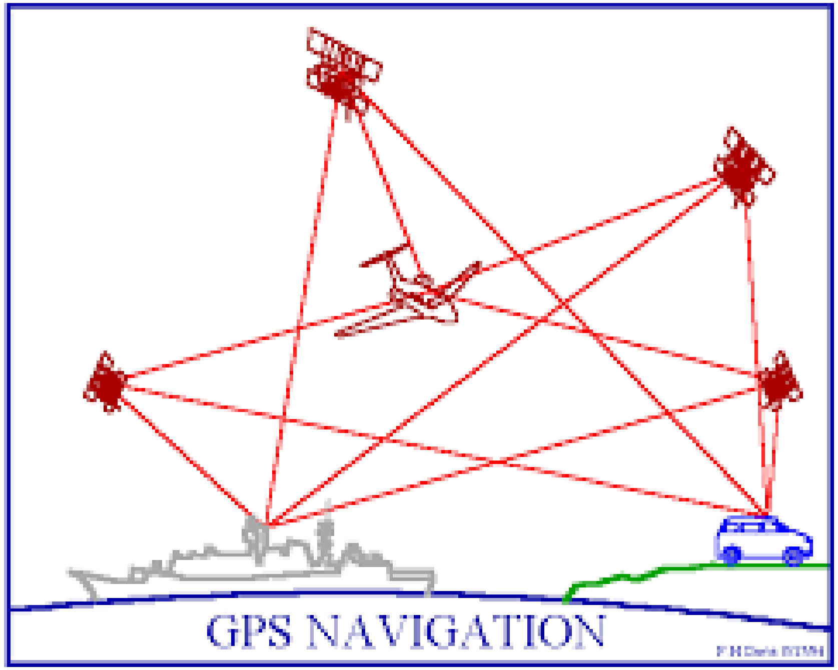

Ashtech

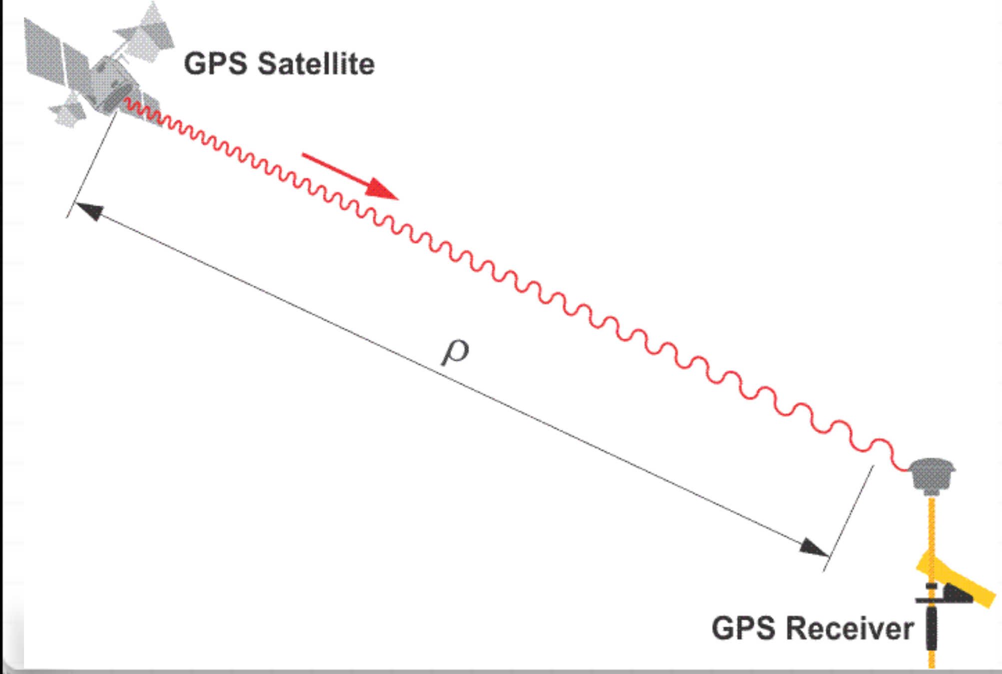
GPS Receivers

- **Most GPS receivers consist of three basic components: (1) an antenna, which receives the signal and, in some cases, has anti-jamming capabilities; (2) a receiver-processor unit, which converts the radio signal to a useable navigation solution; and (3) a control/display unit, which displays the positioning information ...**

- **GPS receivers can be divided into three general classes: survey-grade, mapping-grade and consumer-grade (or recreational-grade).**


GPS Satellites
GPS Signal
GPS Signal
Stationary GPS Receiver
Handset with GPS and GSM
Assistance Information
Base Station
Base Station
MSC
A-GPS Server


GPS NAVIGATION


GPS Satellite
ρ
GPS Receiver

Hand-held Receivers

Satellite Signals

- **Satellites generate two codes:**
  - **- Coarse Acquisition (C/A) available to the public.**
  - **- Code designed mainly for the military use exclusively.**
- **Two types of carrier waves (L1 and L2) at two different wave lengths carry these codes.**
- **Receivers can use carrier phase measurement and / or code measurement. Carrier phase measurement is more accurate than code measurement.**
- **Accuracy of the receivers increases by the increase in number of measurements and frequencies.**

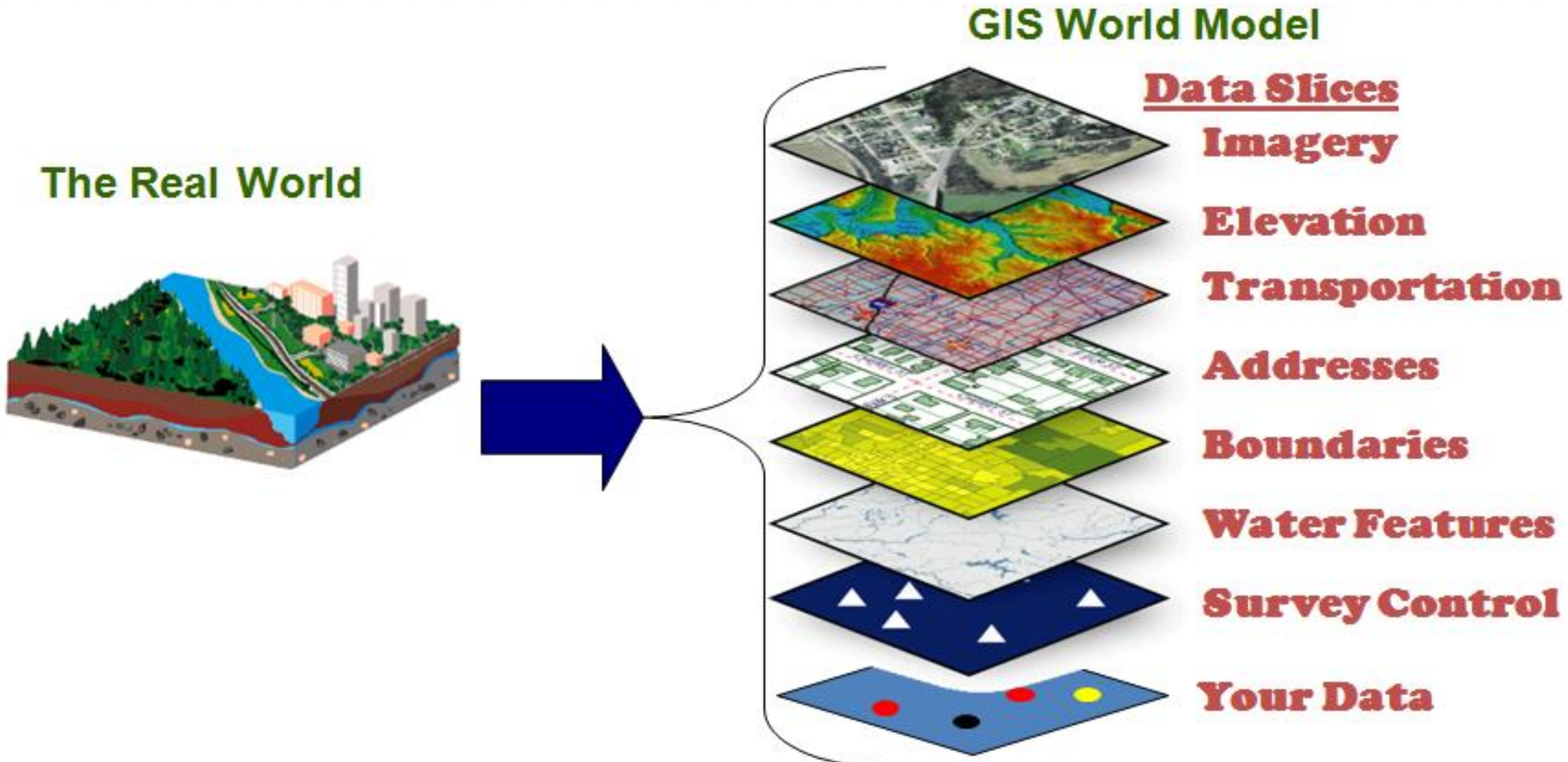
Position Measurements

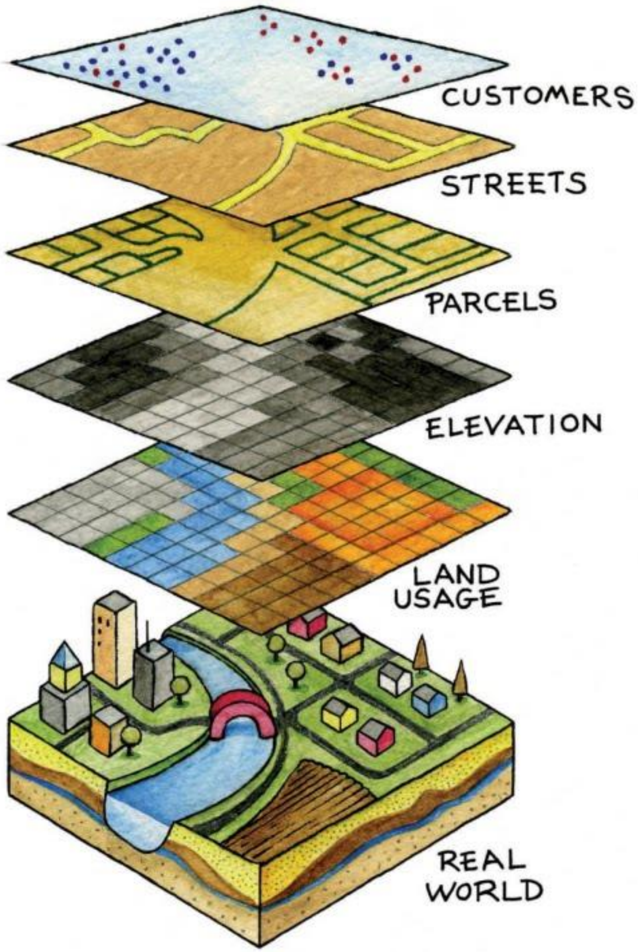
- **Single Point Positioning: uses one GPS receiver to track satellite code signals.**
-
- **Relative Positioning: employs two GPS receivers to track satellite signals to determine baseline vector (DX, DY, and DZ) between both stations. Data should be collected from both receivers by the same set of satellites simultaneously.**

- **Differential Global Positioning System (DGPS): uses two or more GPS receivers to track the same satellites simultaneously. At least one receiver must be set up at a station of known coordinates. Therefore, it consists of one base receiver with radio and a set of rover receivers in a radius < 15 km.**

Geographic Information System (GIS)

- Computerized system for capturing, storing, checking, integrating, manipulating, and displaying data related to positions on the earth's surface.
- GIS uses the GPS for its creation.

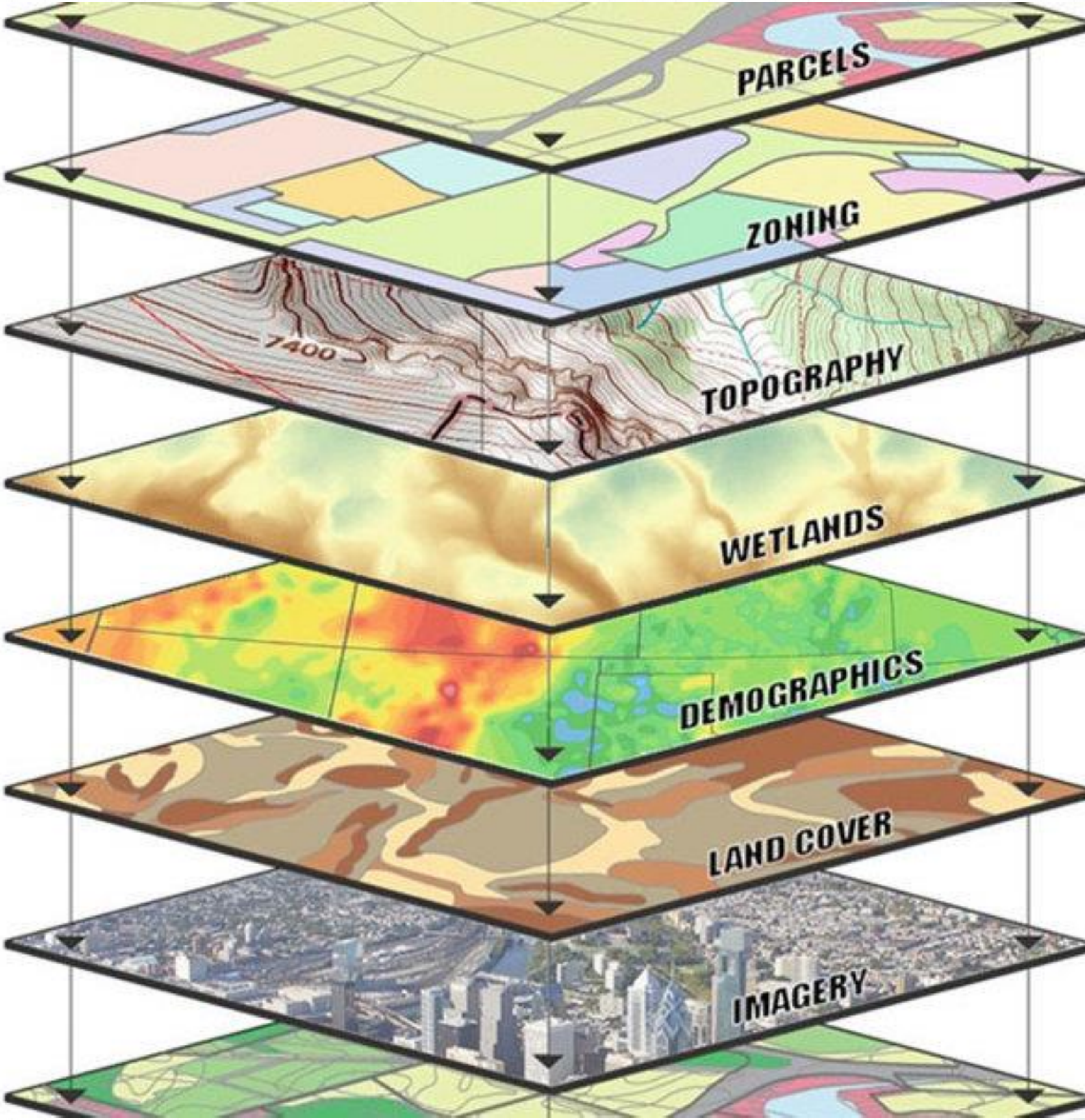




GIS

geographic information system

Different layers of data can be combined through a GIS to represent realistic and integrated digital maps of the Earth's surface (Source: http://www.turfimage.com/)




WHAT IS GIS?
GEOGRAPHIC INFORMATION SYSTEMS
MAPS
APPS
ANALYSIS
CAREERS
DATA
SOFTWARE
SATELLITES

Some of the information provided in this file was collected from:

https://en.wikipedia.org/wiki/Global_Positioning_System

https://gssc.esa.int/navipedia/index.php/GPS_Receivers