# DI03006021: Advance Surveying

GTU Faculty

# Outline I

# Outline II

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## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit I:** Theodolite Traverse
**Lecture Topic:** Introduction to Theodolite, Uses of Theodolite, Sketch and Parts of Transit Vernier Theodolite.

- **Lecture Objectives:**
  - Define a theodolite and explain why it is the most precise angle-measuring surveying instrument.
  - List the principal engineering & field uses of a theodolite.
  - Classify theodolites: transit vs. non-transit, vernier vs. optical/digital.
  - Identify and label the major components of a **Transit Vernier Theodolite** with a neat labeled sketch.
- **Prerequisite Knowledge:** Basic chain/tape surveying, compass surveying, concept of bearings and angles.

# What is a Theodolite? I

## Definition

A **theodolite** is a precision surveying instrument used to measure both **horizontal angles** and **vertical angles** with a very high degree of accuracy, using a telescope that can rotate about a vertical axis and a horizontal (trunnion) axis.

## Why Theodolite over Compass?

- A magnetic compass reads bearings only to the nearest $0.5^\circ$ and is disturbed by local attraction (iron ore, steel structures, power lines).
- A theodolite measures angles directly to $20''$ or $1''$ using graduated circles and verniers/micrometers, independent of magnetic field.
- Suitable for precise traverse surveys, triangulation, setting out curves, and trigonometric levelling.

## Historical Note

Modern transit theodolites evolved from simple graphometers and altazimuth instruments used since the 16th century for astronomical and land-survey angle measurement.

# Uses / Applications of Theodolite I

1. **Measuring horizontal angles** between survey lines for traversing and triangulation.
2. **Measuring vertical angles** for trigonometric levelling (finding elevations of inaccessible points).
3. **Prolonging a survey line** beyond an obstacle and **ranging** intermediate points on a straight line.
4. **Locating points** by the method of intersection and resection.
5. **Setting out engineering works**: building alignment, road/canal centre lines, bridge piers.
6. **Setting out horizontal and transition curves** on roads and railways.
7. **Determining the difference in elevation** between points where direct/fly levelling is impractical.
8. **Measuring magnetic bearings** of lines using the built-in compass box.
9. **Tunnelling and underground alignment** – transferring alignment through shafts using the transiting facility.
10. **Astronomical observations** – determining true meridian, latitude, and azimuth of a line by observing the sun or stars.

# Classification of Theodolites I

## Based on Telescope Motion

- **Transit Theodolite:** The telescope can be revolved (plunged/transited) through a complete $180^\circ$ in the vertical plane about the horizontal axis. Almost all modern theodolites are transit type.
- **Non-Transit Theodolite:** Telescope cannot be transited; largely obsolete today.

## Based on Angle Reading System

- **Vernier Theodolite:** Angles read using a main graduated circle plus a vernier scale (least count typically $20''$). Subject of this course.
- **Micrometer / Optical Theodolite:** Uses an optical micrometer for finer readings (least count $1''$–$10''$).
- **Electronic / Digital Theodolite:** Angles displayed digitally using electronic encoders; often combined with EDM to form a **Total Station**.

## Based on Number of Verniers

- **Simple / Y-theodolite:** Single vernier (telescope mounted in Y-shaped bearings).
- **Vernier Transit Theodolite:** Two verniers (A, B) $180^\circ$ apart on the horizontal plate – this course's focus.

# Parts of a Transit Vernier Theodolite (1/2) I

## Major Components (Bottom to Top) – Part 1

1. **Tripod:** Three adjustable wooden/metal legs supporting the instrument at working height.
2. **Levelling Head (Tribrach):** Two parallel triangular plates connected by three **foot screws** used for levelling; carries the plumbing arrangement (plumb bob hook or optical plummet).
3. **Lower Plate (Scale Plate):** Circular plate graduated from $0^\circ$ to $360^\circ$; rotated/clamped using the **lower clamp and tangent screw**.
4. **Upper Plate (Vernier Plate):** Carries two verniers (A and B) diametrically opposite; rotates independently of the lower plate using the **upper clamp and tangent screw**.
5. **Standards (A-frame):** Two vertical frames rising from the upper plate that support the horizontal (trunnion) axis of the telescope.
6. **Telescope:** Mounted on the trunnion axis between the standards; provides line of sight, fitted with objective lens, eyepiece, and cross-hair diaphragm.

## Major Components (Bottom to Top) – Part 2

7. **Vertical Circle:** Graduated circle attached to and rotating with the telescope's trunnion axis, used to measure vertical angles.
8. **Index Frame (T-frame / Vernier Frame):** Carries verniers C and D for reading the vertical circle, along with the **altitude bubble (clip bubble)**.
9. **Plate Levels:** Sensitive bubble tubes mounted on the upper plate, used to level the instrument's vertical axis.
10. **Compass Box:** Mounted on the upper plate to read the magnetic bearing of the line of sight.
11. **Clamps and Tangent Screws:** Upper clamp/tangent, lower clamp/tangent, and telescope (vertical) clamp/tangent – provide coarse fixing and fine adjustment of rotation.
12. **Shifting Head:** Plate between tribrach and lower plate allowing small lateral shifts for precise centring over the station.

# Labeled Sketch of Transit Vernier Theodolite I

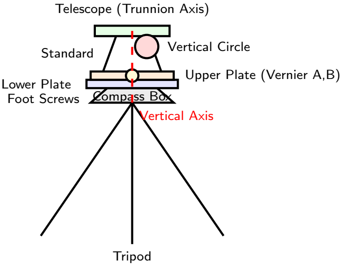



Figure: Schematic sketch of a Transit Vernier Theodolite (major parts labeled)

# Summary & GTU Exam Focus I

## Key Takeaways

- Theodolite = most precise angle-measuring instrument, gives both horizontal and vertical angles.
- Transit theodolite: telescope can be plunged $180^\circ$ about the horizontal axis.
- Vernier theodolite reads angles via main scale + vernier (LC = $20''$ typically).
- Major parts: tribrach, lower plate, upper plate, standards, telescope, vertical circle, T-frame, plate levels, compass box, clamps/tangent screws.

## GTU Exam Important Questions

1. Define theodolite. State any six uses of a theodolite in engineering surveys.
2. Draw a neat labeled sketch of a transit vernier theodolite.
3. Differentiate between transit and non-transit theodolite.
4. Distinguish between vernier theodolite and micrometer/electronic theodolite.

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit I:** Theodolite Traverse
**Lecture Topic:** Reading of Main & Vernier Scales (Horizontal & Vertical Plates) and Temporary Adjustment of Transit Theodolite.

- **Lecture Objectives:**
  - Master the principles of reading main scale and vernier scale on transit theodolites.
  - Derive and calculate the **Least Count (LC)** of horizontal and vertical plates.
  - Understand the role of dual verniers (A & B, C & D) in eliminating instrumental errors.
  - Systematically perform the four stages of **Temporary Adjustment** at a station.
  - Eliminate parallax error by proper eyepiece and objective focusing.
- **Prerequisite Knowledge:** Basics of transit theodolite components (lower plate, upper plate, foot screws, plate bubbles, telescope).

# Scales on Transit Theodolite: Horizontal & Vertical I

## Horizontal Circle Plate System

The horizontal circle measures horizontal angles and azimuths. It consists of two concentric plates:

- **Lower Plate (Main Scale Plate):** Beveled edge graduated from $0^\circ$ to $360^\circ$ clockwise. Each degree is typically subdivided into 3 equal parts ($20'$ divisions) or 4 equal parts ($15'$ divisions).
- **Upper Plate (Vernier Plate):** Carries two vernier scales designated as **Vernier A** and **Vernier B**, located diametrically opposite ($180^\circ$ apart).

## Vertical Circle Plate System

The vertical circle measures vertical angles (angles of elevation or depression):

- Attached directly to the trunnion (transverse) axis of the telescope, rotating with it.
- Graduated into four quadrants from $0^\circ$ to $90^\circ$ in opposite directions, or $0^\circ$ to $360^\circ$ continuously.
- Carries two vernier scales designated as **Vernier C** and **Vernier D** attached to the T-frame (index arm).

# Least Count (LC) of Theodolite Vernier I

## Definition of Least Count

The **Least Count (LC)** is the smallest value of angle that can be measured directly and accurately using the vernier scale without estimation.

## General Formula

$$\text{Least Count (LC)} = \frac{s}{n} = s - v$$

where:

- $s$ = Value of one smallest division on Main Scale
- $v$ = Value of one division on Vernier Scale
- $n$ = Total number of divisions on Vernier Scale

## Standard 20″ Transit Theodolite Derivation

- Smallest Main Scale division ($s$) $= 20' = 20 \times 60'' = 1200''$.
- Total Vernier Scale divisions ($n$) $= 60$ divisions (matching 59 main scale divisions).
- Therefore,
$$\text{LC} = \frac{s}{n} = \frac{20'}{60} = \frac{1}{3}' = \left(\frac{1}{3} \times 60\right)'' = 20'' \text{ (20 seconds of arc)}$$

1. **Identify Main Scale Reading (MSR):**
   - Observe the index mark (0 of Vernier). Look at the main scale line just preceding the Vernier zero mark.
   - Record degrees and primary fractional parts (e.g., $42^\circ 40'$).
2. **Locate Coinciding Vernier Division (VSR):**
   - Look along the vernier scale to find the division line that aligns perfectly with a main scale line.
   - Count the division number ($n_c$) from the vernier index (0).
3. **Calculate Fractional Reading:**
   - Multiply coinciding division number ($n_c$) by the Least Count ($20''$).
   - Vernier Reading $= n_c \times 20''$.
4. **Compute Total Angle Reading:**
   - Total Reading $= \text{MSR} + (n_c \times \text{LC})$.

Table: Numerical Example of Scale Reading

| Component | Observed Value | Value in Degrees/Minutes/Seconds |
|---|---|---|
| Main Scale Index | Passed $48^\circ 20'$ mark | $48^\circ 20' 00''$ |
| Coinciding Vernier Line | $17^{\text{th}}$ division mark | $17 \times 20'' = 340'' = 5'40''$ |
| **Final Observed Reading** | **MSR + Vernier Value** | $\mathbf{48^\circ 25' 40''}$ |

# TikZ Illustration: Vernier Scale Coincidence I

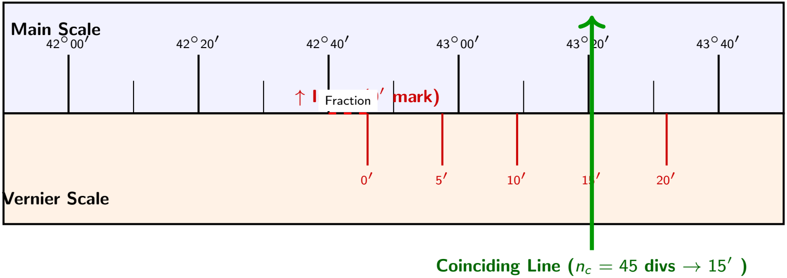



Figure: Conceptual Vernier Coincidence on a Transit Theodolite Circle

# Role of Dual Verniers (A & B, C & D) I

## Why Two Verniers on Opposite Sides?

Due to manufacturing imperfections, the center of the inner axis (vernier plate) may not coincide perfectly with the center of the outer axis (main scale plate). This creates an **eccentricity error**.

- **Horizontal Plate Verniers (Vernier A and Vernier B):**
  - Vernier A provides the primary angle reading ($0^\circ$ to $360^\circ$).
  - Vernier B is positioned exactly $180^\circ$ opposite to Vernier A.
  - If no eccentricity error exists: Reading B = Reading A + $180^\circ$.
  - If eccentricity exists, Vernier A and B readings differ slightly from $180^\circ$.
  - **Corrected Horizontal Angle:**

$$\text{Corrected Reading} = \frac{\text{Reading A} + (\text{Reading B} \mp 180^\circ)}{2}$$

- **Vertical Plate Verniers (Vernier C and Vernier D):**
  - Vernier C and D are located at opposite ends of the vertical index arm.
  - Mean of Vernier C and D eliminates eccentricity of the vertical scale axis.

# Introduction to Temporary Adjustment I

## What is Temporary Adjustment?

**Temporary adjustments** are the set of operations that must be performed at *every survey station* before taking any angular or distance measurements with the theodolite.

## Objective of Temporary Adjustment

To set up the instrument over the station mark, make its vertical axis truly vertical, eliminate parallax, and ensure accurate, repeatable horizontal and vertical observations.

## Four Standard Steps in Proper Sequence

1. **Setting Up:** Spreading tripod legs and rough positioning over station.
2. **Centering:** Placing vertical axis precisely over the ground station mark.
3. **Leveling Up:** Aligning vertical axis truly vertical using foot screws and plate level bubbles.
4. **Elimination of Parallax:** Adjusting eyepiece and objective focus for sharp image formation on cross-hairs.

# Step 1 & 2: Setting Up and Centering I

## Step 1: Setting Up the Instrument

- Place tripod approximately over the ground station mark at a comfortable working height.
- Spread tripod legs firmly apart, pushing leg shoes firmly into firm ground.
- Ensure tribrach head is roughly horizontal by visual inspection.
- Attach the transit theodolite securely to the tripod head using central fixing screw.

## Step 2: Centering the Theodolite

- **Definition:** Operation of bringing the vertical axis of the instrument directly over the ground station mark.
- **Using Plumb Bob:**
  - Suspend a plumb bob from the hook beneath the central axis screw.
  - Adjust tripod legs by shifting them radially or circumferentially until plumb tip points exactly to station mark.
- **Using Optical / Laser Plummet:**
  - Modern transit theodolites feature an optical plummet in the tribrach.
  - Look through optical plummet eyepiece and shift instrument head on shifting head plate until cross-mark coincides with station peg.

## Principle of Leveling Up

Leveling brings the vertical rotation axis of the transit theodolite into a truly vertical line, ensuring the horizontal plate is perfectly horizontal.

## Detailed 5-Step Procedure

1. **Position 1 (Parallel to 2 Screws):** Turn upper plate until plate bubble tube is parallel to the line joining any two foot screws (say Screws A and B).
2. **Manipulate Screws A & B:** Turn both foot screws A and B simultaneously in opposite directions—either **both INWARDS** (thumbs towards each other) or **both OUTWARDS** (thumbs away from each other)—until bubble centers.
3. **Position 2 (90° Rotation):** Rotate upper plate through 90° so bubble tube lies perpendicular to line AB (over the third foot screw C).
4. **Manipulate Screw C:** Turn ONLY foot screw C until bubble comes to center.
5. **Verification:** Rotate back to Position 1 and re-check. Repeat steps until bubble stays centered in ALL 360° positions of rotation.

# TikZ Illustration: Leveling Up Positions I

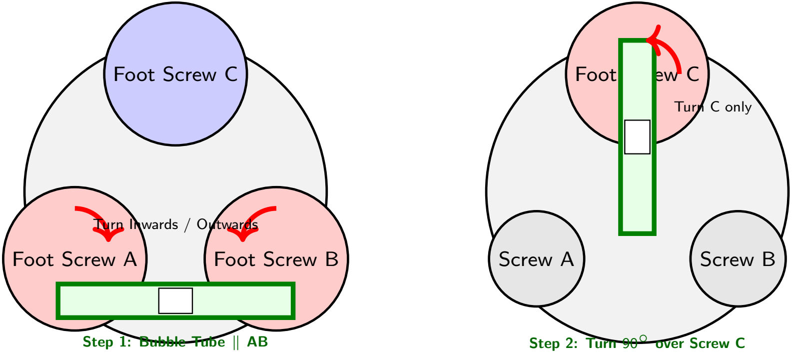



Figure: Foot Screw Orientation during Leveling Up Procedure

## What is Parallax in Surveying Telescope?

**Parallax** is a condition in which the image formed by the objective lens does not lie strictly in the plane of the cross-hairs (reticle).

- Causes apparent movement between cross-hairs and target image when observer shifts eye position.
- Introduces severe angular measurement errors if not completely eliminated.

## Two Steps to Eliminate Parallax

1. **Focusing the Eyepiece (For Sharp Cross-Hairs):**
    - Point telescope towards sky or clear white sheet of paper.
    - Rotate or slide eyepiece in/out until cross-hairs appear perfectly sharp, clear, and jet black.
2. **Focusing the Objective Lens (For Sharp Target Image):**
    - Direct telescope towards target/ranging rod.
    - Rotate external focusing screw until clear image of target is formed on reticle plane.
    - Move eye up and down slightly; if no relative motion exists between target and cross-hair, parallax is fully eliminated!

# Comparison & GTU Exam Focus I

Table: Temporary vs Permanent Adjustments

| Parameter | Temporary Adjustment | Permanent Adjustment |
|---|---|---|
| **Frequency** | Done at *every setup* station. | Done periodically or after instrument repair. |
| **Objective** | Setup instrument, level vertical axis, focus target. | Fix internal fundamental lines (line of sight, altitude bubble axis, trunnion axis). |
| **Operator** | Performed directly by field surveyor. | Performed by instrument technician / manufacturer. |

## GTU Exam Important Questions

1. Define Least Count of a theodolite and derive its value ($20''$).
2. Enlist temporary adjustments of transit theodolite in correct sequence.
3. Explain 3-foot screw method of leveling up a theodolite with neat diagram.
4. What is parallax? How is it eliminated during field observations?

# Lecture 03: Agenda & Recap I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit I:** Theodolite Traverse
**Lecture Topic:** Permanent Adjustment of Theodolite (Fundamental Axes and their Relationship); Definitions and Technical Terms.

## Recap of Lecture 02

Lecture 02 covered **Temporary Adjustments** – setting up, centring, levelling, and elimination of parallax – performed *at every station*. Today we study **Permanent Adjustments**, which correct the internal geometry of the instrument itself and are *not* required at every setup.

- **Lecture Objectives:**
    - Identify the four fundamental axes of a transit theodolite.
    - State the desired geometrical relationships between these axes.
    - Explain the test-and-correct procedure for each permanent adjustment.
    - Define key technical terms used in theodolite work.

1. **Vertical Axis (VV):** The axis about which the upper plate and telescope rotate in the horizontal plane. Must be truly vertical (achieved by temporary adjustment / plate levels).
2. **Horizontal Axis / Trunnion Axis (HH):** The axis about which the telescope rotates in the vertical plane, supported by the two standards.
3. **Line of Collimation (Line of Sight, LC):** The imaginary line joining the intersection of cross-hairs to the optical centre of the objective lens, extended to the object.
4. **Axis of the Plate Level / Axis of the Altitude Level:** The horizontal tangent to the bubble tube curve at the centre of its run, when the bubble is centred.

## Why These Axes Matter

Accurate angle measurement requires all four axes to bear a fixed, correct geometrical relationship to one another. Any deviation is called **maladjustment**, producing systematic instrumental errors that permanent adjustment removes.

# Diagram: Mutual Relationship of the Axes I

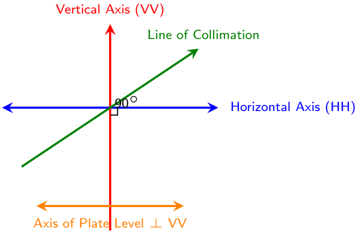



Figure: Idealised mutual relationship: HH ⊥ VV, Line of Collimation ⊥ HH, Plate Level Axis ⊥ VV

# Desired Relationships Between Axes I

Table: Fundamental Adjustment Conditions

| **Adjustment** | **Required Condition** |
|---|---|
| 1. Plate Level Adjustment | Axis of each plate level must be perpendicular to the vertical axis (VV). |
| 2. Line of Collimation Adjustment | Line of collimation must be perpendicular to the horizontal axis (HH), i.e. it must lie exactly along the optical axis. |
| 3. Horizontal Axis Adjustment (Spire Test) | The horizontal/trunnion axis (HH) must be exactly perpendicular to the vertical axis (VV). |
| 4. Altitude Bubble & Vertical Index Adjustment | Axis of altitude bubble must be parallel to line of collimation; vertical circle vernier must read exactly $0^\circ$ when line of collimation is horizontal. |

## Consequence of Maladjustment

If HH is not perpendicular to VV, sighting a high object and swinging the telescope down traces a line that departs from the vertical plane – causing errors in transferred alignment (e.g. tall building/tower work).

# Adjustment 1: Plate Levels I

## Test

1. Level the instrument approximately and centre the plate bubble using the foot screws.
2. Rotate the upper plate through exactly $180^\circ$ about the vertical axis.
3. If the bubble remains centred, the axis is in adjustment. If it moves off centre, maladjustment exists.

## Correction

1. Bring the bubble halfway back towards the centre using the **foot screws**.
2. Bring the remaining half back using the **capstan-headed adjusting screws** of the bubble tube itself.
3. Repeat the $180^\circ$ test until the bubble stays centred in every position of the upper plate.

# Adjustment 2: Line of Collimation I

## Objective

To make the line of collimation coincide with the true optical axis of the telescope (perpendicular to the horizontal axis).

## Test & Correction (Reversal Method)

1. Set up the theodolite midway between two pegs $A$ and $B$, about 100–150 m apart.
2. With face left, sight and mark point $A_1$ on a target/peg at $A$; transit the telescope and mark $B_1$ at $B$.
3. Change face (face right); sight $A$ again to get $A_2$, transit and mark $B_2$ at $B$.
4. If the line of collimation is correct, $B_1$ and $B_2$ coincide. Any discrepancy indicates collimation error.
5. **Correction:** Mark a point $B_3$ at one-quarter of the distance $B_1B_2$ from $B_2$ (nearer the correct final position); shift the diaphragm cross-hair (using its adjusting screws) until the line of sight bisects $B_3$.

# Adjustment 3: Horizontal Axis (Spire Test) I

## Objective

To make the horizontal (trunnion) axis exactly perpendicular to the vertical axis.

## Test Procedure (Spire Test)

1. Set up the theodolite near a tall, well-defined vertical structure (spire, flagpole, or building edge) at a steep sighting angle.
2. With face left, sight the high point $P$ of the spire and clamp; depress (plunge) the telescope to the ground level and mark point $Q_1$.
3. Change face; again sight $P$ with face right, depress the telescope, and mark point $Q_2$.
4. If HH $\perp$ VV, $Q_1$ and $Q_2$ coincide. If not, the horizontal axis is maladjusted.

## Correction

Raise or lower one end of the horizontal axis using the adjustable screw on one of the standard bearings until the telescope, when sighted to a point midway between $Q_1 Q_2$ and elevated back to $P$, exactly bisects $P$. This is normally a workshop/manufacturer-level adjustment.

## Objective

(a) Axis of the altitude bubble parallel to the line of collimation; (b) vertical circle vernier reads exactly $0^\circ$ when the line of collimation is horizontal.

## Test & Correction

1. Level the instrument and take a reading on a levelling staff with the altitude bubble centred (face left); record staff reading $R_1$ and vertical circle reading (should be $0^\circ$).
2. Change face and repeat; record $R_2$ and the vertical circle reading.
3. If the mean of the face-left and face-right vertical circle readings is not exactly $0^\circ$ (for a horizontal line of sight), a **vertical index error** exists.
4. **Correction:** Use the altitude bubble's tangent screw to bring the vernier exactly to $0^\circ$; then re-centre the bubble using its own capstan adjusting screws (not the tangent screw).

## Field Practice

Even without adjustment, vertical index error can be **eliminated computationally** by taking the mean of face-left and face-right vertical angle readings – this is why vertical angles are always observed on both faces.

# Definitions & Technical Terms I

- **Centring:** Setting the vertical axis of the instrument exactly over the ground survey station mark.
- **Transiting (Plunging / Reversing):** Rotating the telescope through $180^\circ$ in the vertical plane about the horizontal axis.
- **Swinging the Telescope:** Rotating the telescope about the vertical axis; *swinging right* = clockwise, *swinging left* = anticlockwise.
- **Face Left (Face Normal):** Observation with the vertical circle to the *left* of the observer/telescope.
- **Face Right (Face Inverted):** Observation with the vertical circle to the *right*, obtained after transiting and swinging $180^\circ$.
- **Changing Face:** The process of converting a face-left observation to a face-right observation (transit + swing $180^\circ$).
- **Line of Sight (Line of Collimation):** Line joining intersection of cross-hairs and optical centre of objective, extended to the object point.
- **Axis of the Telescope:** Line joining the optical centre of the objective lens to the centre of the eyepiece.
- **Height of Instrument (in angular work):** Elevation of the trunnion axis above the ground station (distinct from the levelling "height of instrument").
- **Face Left / Face Right Mean:** Average of the two face readings, used to eliminate several instrumental errors automatically.

# Summary & GTU Exam Focus I

## Key Takeaways

- Permanent adjustments correct internal instrument geometry; unlike temporary adjustments, they are not required at every station.
- Four essential relationships: plate level axis $\perp$ VV; line of collimation $\perp$ HH; HH $\perp$ VV; altitude bubble axis $\parallel$ line of collimation with zero vertical index error.
- Reversal (double-sighting) and spire tests are the standard field tests; corrections use the instrument's own adjusting screws in defined sequence.
- Observing both face left and face right and taking the mean eliminates most residual instrumental errors without physical adjustment.

## GTU Exam Important Questions

1. List the fundamental axes of a transit theodolite and state their desired mutual relationships.
2. Describe the reversal method of testing and correcting the line of collimation.
3. Explain the spire test for adjustment of the horizontal axis.
4. Define: transiting, swinging the telescope, face left, face right, changing face.

# Lecture 04: Agenda & Course Context I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit I:** Theodolite Traverse
**Lecture Topic:** Methods of Measuring Horizontal Angles and Vertical Angles.

- **Lecture Objectives:**
  - Explain the ordinary (direct) method of measuring a horizontal angle.
  - Perform the **method of repetition** to measure a horizontal angle with increased precision.
  - Perform the **method of reiteration (directional method)** for measuring several angles from one station.
  - Measure vertical angles correctly, eliminating index error by the mean-of-two-faces technique.

# Ordinary (Direct) Method of Measuring a Horizontal Angle I

## Objective

To measure the horizontal angle $\angle AOB$ between two rays $OA$ and $OB$ from station $O$.

## Procedure

1. Set up and level the theodolite at $O$ (temporary adjustment); keep both upper and lower clamps free initially.
2. Set the horizontal circle reading to $0^\circ 00' 00''$ (or any convenient value) and clamp the upper plate; sight the first point $A$ using the lower clamp & tangent screw, bisecting it exactly.
3. Release the upper clamp; swing the telescope clockwise to sight the second point $B$; use upper clamp & tangent screw for exact bisection.
4. Read both verniers A and B; take the mean of the two vernier readings to eliminate eccentricity error of the circle.
5. The angle $\angle AOB =$ final reading at $B -$ initial reading at $A$.

# Diagram: Horizontal Angle Observation I

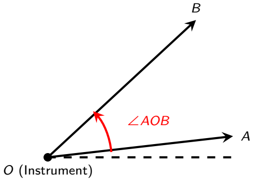



Figure: Horizontal angle between rays $OA$ and $OB$

# Method of Repetition I

## Principle

The horizontal angle is measured several times (repetitions), accumulating the readings on the circle itself, then dividing the final accumulated reading by the number of repetitions – this averages out reading (vernier) errors.

## Procedure (Face Left)

1. Set the vernier to $0^\circ$; bisect $A$ using the lower clamp/tangent screw.
2. Release upper clamp, swing to $B$, bisect using upper clamp/tangent screw; record reading = angle $AOB$ (first observation).
3. **Without touching the upper plate**, release the lower clamp, swing back to $A$ (the circle now carries the previous reading), and re-bisect $A$ using the lower tangent screw.
4. Release upper clamp, again swing and re-bisect $B$; the circle reading now shows *twice* the angle.
5. Repeat for 3 (or more) repetitions; record final accumulated reading.

$$\text{Mean Angle} = \frac{\text{Final accumulated reading}}{\text{Number of repetitions}}$$

## Face Right Repetitions

Repeat the same number of repetitions with face right (changed face) to eliminate instrumental errors; take overall mean of both faces.

# Worked Example: Method of Repetition I

## Problem

The angle $AOB$ was measured by repetition, three times on face left and three times on face right. The final vernier reading after 3 repetitions on face left was $124^\circ 30' 00''$, and after 3 repetitions on face right was $124^\circ 29' 40''$ (each starting fresh from $0^\circ$). Find the most probable value of the angle.

## Solution

$$\text{Mean angle (Face Left)} = \frac{124^\circ 30' 00''}{3} = 41^\circ 30' 00''$$

$$\text{Mean angle (Face Right)} = \frac{124^\circ 29' 40''}{3} = 41^\circ 29' 53.3''$$

$$\text{Most Probable Value} = \frac{41^\circ 30' 00'' + 41^\circ 29' 53.3''}{2} = \mathbf{41^\circ 29' 56.7''}$$

## Note

Method of repetition eliminates: (a) errors of graduation on the circle, by using different parts of the circle for each set; (b) errors in bisecting the target, by averaging multiple readings.

# Method of Reiteration (Directional Method) I

## When Used

Used when **several angles** are to be measured from a single station (e.g. triangulation station with multiple rays $A$, $B$, $C$, $D$).

## Procedure

1. Set the vernier to $0^\circ 00' 00''$ and bisect the first (reference) station $A$.
2. Release upper clamp only; successively sight $B$, $C$, $D$ *clockwise*, recording the horizontal circle reading at each – **without resetting to zero between sights**.
3. Finally, continue swinging clockwise back to close the horizon on $A$ again – the closing reading should return to $0^\circ 00' 00''$ (or reveal small **swing/closing error**).
4. Change face (face right) and repeat the entire round in the *reverse* (anticlockwise) direction back to $A$.
5. The angle between any two rays = difference of their respective mean circle readings (face left and face right mean).

## Advantage

A single reference (zero) direction serves all angles at that station – faster than repeating full setups for every pair of rays, and any closing error is immediately visible.

# Measurement of Vertical Angles I

## Definition

A **vertical angle** is the angle, measured in a vertical plane, between the line of sight to a point and the horizontal plane – positive (angle of elevation) if the point is above the horizontal, negative (angle of depression) if below.

## Procedure

1. Level the instrument carefully; ensure altitude bubble is centred (or use automatic vertical-circle compensator if fitted).
2. With face left, sight the target and clamp the vertical circle; make final bisection with the vertical tangent screw.
3. Read verniers C and D of the vertical circle; take the mean.
4. Change face (face right); resight the same target and again read verniers C, D; take the mean.
5. **Final Vertical Angle** = mean of the face-left and face-right observations – this automatically cancels any **vertical index error**.

$$\text{Vertical Angle} = \frac{(\text{Face Left Reading}) + (\text{Face Right Reading})}{2}$$

# Summary & GTU Exam Focus I

## Key Takeaways

- Ordinary method: single set of readings at $A$ and $B$; quick but least precise.
- Repetition method: same angle measured repeatedly, accumulated on the circle, then averaged – eliminates graduation and bisection errors.
- Reiteration (directional) method: multiple angles from one station referenced to a common zero – efficient for triangulation stations.
- Vertical angles are always the mean of face-left and face-right readings to cancel index error.

## GTU Exam Important Questions

1. Describe the method of repetition for measuring a horizontal angle with a numerical example.
2. Explain the method of reiteration and state its advantage over repeated setups.
3. How is a vertical angle measured and how is the vertical index error eliminated?

# Lecture 05: Agenda & Course Context I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit I:** Theodolite Traverse
**Lecture Topic:** Using a Theodolite to Measure a Magnetic Bearing, Prolong a Line, and Range a Line; Measuring Direct and Deflection Angles.

- **Lecture Objectives:**
  - Measure the magnetic bearing of a line using the theodolite's compass box.
  - Prolong a straight survey line beyond an obstacle using double-sighting.
  - Range intermediate points on a straight line, directly and by reciprocal ranging.
  - Distinguish direct angles from deflection angles and measure each correctly.

# Measuring the Magnetic Bearing of a Line I

## Procedure

1. Set up and level the theodolite at station $A$; loosen the compass needle clamp inside the compass box.
2. Loosen the upper clamp and rotate the upper plate until the compass needle (or the graduated compass ring) settles and the $0^\circ$–$360^\circ$ N–S line of the compass box is aligned with magnetic north.
3. Clamp the compass ring / note the reading; sight the target point $B$ using upper clamp & tangent screw.
4. The circle reading directly gives the **Fore Bearing (FB)** of line $AB$, measured clockwise from magnetic north ($0^\circ$ to $360^\circ$, Whole Circle Bearing system).

## Fore Bearing – Back Bearing Relationship

$$\text{Back Bearing (BB)} = \text{Fore Bearing (FB)} \pm 180^\circ$$

(Add $180^\circ$ if FB $< 180^\circ$; subtract $180^\circ$ if FB $> 180^\circ$.)

## Note

Bearings observed with the theodolite compass are approximate (nearest $0.5^\circ$) and are mainly used as a check against gross errors in angle measurement, not as the primary control.

# Prolonging a Straight Line Using a Theodolite I

## Objective

To extend a straight line $AB$ beyond $B$ to a new point $C$ such that $A$, $B$, $C$ are exactly collinear – essential when direct ranging by eye is obstructed or long-range precision is required.

## Procedure (Double Sighting / Double Centring Method)

1. Set up the theodolite at $B$, on line $AB$; with **face left**, backsight $A$ accurately and clamp.
2. Transit (plunge) the telescope; this projects the line of sight forward. Mark a point $C_1$ along this forward direction.
3. Change face to **face right**; again backsight $A$ and transit the telescope forward; mark point $C_2$.
4. Due to instrumental maladjustment (line of collimation / horizontal axis errors), $C_1$ and $C_2$ may not coincide exactly.
5. The correct point $C$ is taken as the **midpoint of** $C_1C_2$, eliminating the effect of instrumental errors.

# Diagram: Prolonging Line $AB$ to $C$ I

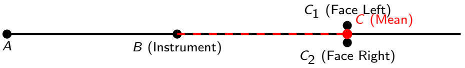



Figure: Midpoint of $C_1 C_2$ eliminates instrumental errors in prolongation

## Direct Ranging

1. Set up the theodolite at one end station $A$ of the line, sight the far end station $B$, and clamp.
2. Direct assistants holding ranging rods to move sideways until each rod is exactly bisected by the vertical cross-hair – this fixes intermediate points precisely on line $AB$.
3. Faster and more accurate than ranging by eye alone, especially over long distances.

## Indirect / Reciprocal Ranging

Used when the two end stations $A$ and $B$ are **not intervisible** (e.g. a hill obstructs the line of sight) but each is visible from points near the line.

1. Two theodolites are set up at trial points $M_1$ (near line, visible to $A$) and $N_1$ (near line, visible to $B$).
2. Surveyor at $M_1$ sights $A$ and directs surveyor at $N_1$ to move onto line $M_1A$-extended; surveyor at $N_1$ sights $B$ and directs $M_1$ similarly.
3. By successive mutual approximation (each adjusting to the other's line), both points converge onto the true line $AB$.

# Direct Angles and Deflection Angles I

## Direct (Included) Angle

The angle measured directly between the **back line** and the **forward line** at a traverse station, read clockwise from $0^\circ$ to $360^\circ$ using the ordinary method (backsight, then swing to foresight).

## Deflection Angle

The angle between the **forward prolongation of the previous line** and the **new line**, measured either to the **right (clockwise, R)** or **left (anticlockwise, L)**, ranging from $0^\circ$ to $180^\circ$.

$$\text{Deflection Angle} = 180^\circ - \text{Included Angle}$$

## Field Procedure for Deflection Angle

1. At station $B$ (having come from $A$), backsight $A$ with the vernier set to $0^\circ$.
2. Transit (plunge) the telescope – this points along the forward prolongation of $AB$.
3. Swing to sight the next station $C$; the circle reading directly gives the deflection angle, and its direction (R or L) is noted from which side the swing occurred.

# Diagram: Deflection Angle at Station $B$ I

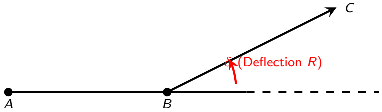



Figure: Deflection angle $\delta$ measured from prolongation of $AB$ to new line $BC$

# Worked Example: Computing Deflection Angles from Bearings I

## Problem

The whole circle bearings (WCB) of the successive traverse lines of a closed traverse *ABCDA* are: $AB = 45^\circ$, $BC = 120^\circ$, $CD = 200^\circ$, $DA = 300^\circ$. Determine the deflection angle at each intermediate station.

## Solution: Deflection = Change in Bearing (R if increasing, L if decreasing, taking the smaller angle)

| Station | Incoming/Outgoing WCB | Change | Deflection Angle |
|---|---|---|---|
| $B$ | $AB = 45^\circ \to BC = 120^\circ$ | $120 - 45 = 75^\circ$ | $75^\circ$ R |
| $C$ | $BC = 120^\circ \to CD = 200^\circ$ | $200 - 120 = 80^\circ$ | $80^\circ$ R |
| $D$ | $CD = 200^\circ \to DA = 300^\circ$ | $300 - 200 = 100^\circ$ | $100^\circ$ R |

## Check

For a closed traverse, $\Sigma$(Right deflections) $-$ $\Sigma$(Left deflections) $= 360^\circ$ (all deflections here are to the right, summing to $255^\circ$ for these three stations; the remaining deflection at $A$ closes the figure).

## Key Takeaways

- Magnetic bearing is read directly off the compass box after aligning it to magnetic north.
- Prolonging a line uses double-sighting (face left/right) and takes the mean point to cancel instrumental error.
- Direct ranging uses cross-hair bisection of ranging rods; reciprocal ranging is used when end stations are not intervisible.
- Direct (included) angle vs. deflection angle: deflection $= 180^\circ -$ included angle, always noted with direction R or L.

## GTU Exam Important Questions

1. Describe the double-sighting method of prolonging a line with a theodolite.
2. Explain reciprocal ranging with a neat sketch, stating when it is used.
3. Differentiate between direct (included) angle and deflection angle.
4. Given whole circle bearings of traverse lines, compute the deflection angle at each station.

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit I:** Theodolite Traverse
**Lecture Topic:** Errors in Theodolite Work; Theodolite Traversing.

- **Lecture Objectives:**
  - Classify the sources of error in theodolite observations: instrumental, personal, and natural.
  - Explain how face-change (double-face) and repetition observations eliminate specific errors.
  - Define theodolite traversing and its classification.
  - Describe the field methods of running a closed theodolite traverse.

## Definition

Errors arising from imperfections or maladjustment in the manufacture/adjustment of the instrument itself.

- **Error due to imperfect adjustment of plate levels:** Vertical axis not truly vertical.
- **Error due to line of collimation not perpendicular to horizontal axis.**
- **Error due to horizontal axis not perpendicular to vertical axis** (affects readings when sighting at high/low vertical angles).
- **Eccentricity of inner and outer axes** of the upper and lower plates – eliminated by reading both verniers (A and B) and taking the mean.
- **Eccentricity of verniers** with respect to the graduated circle centre.
- **Error due to inaccurate graduations** on the horizontal/vertical circles.
- **Vertical index error** – vertical circle vernier does not read $0^\circ$ when line of sight is horizontal.

# Personal Errors and Natural Errors I

## Personal Errors (Observer-Induced)

- Inaccurate centring of the instrument over the station.
- Imperfect levelling (bubble not exactly centred).
- Inaccurate bisection of the target/ranging rod (parallax not fully eliminated).
- Mistake in reading verniers or in booking observations.
- Manipulating wrong tangent screw (clamp not properly engaged before using tangent screw).

## Natural Errors (Environment-Induced)

- High wind causing vibration of the tripod/instrument.
- Unequal atmospheric refraction (temperature gradients bending the line of sight).
- Irregular settlement of the tripod on soft ground during observation.
- Extreme temperature variation causing differential expansion of instrument parts.

# Elimination of Errors: Standard Field Techniques I

Table: Techniques to Eliminate Common Theodolite Errors

| Error Type | Technique | Effect |
|---|---|---|
| Eccentricity of verniers/axes | Read both verniers A & B, take mean | Eccentricity cancelled |
| Line of collimation & horizontal axis maladjustment | Change face (Face Left + Face Right mean) | Instrumental error cancelled |
| Graduation & bisection errors | Method of repetition | Errors averaged out |
| Vertical index error | Mean of FL and FR vertical angles | Index error cancelled |
| Inaccurate centring/levelling | Careful temporary adjustment, optical plummet | Error minimised at source |

## Golden Rule

Always observe angles on **both face left and face right** and take the mean – this single practice eliminates almost all significant instrumental errors without requiring physical adjustment in the field.

# Introduction to Theodolite Traversing I

## Definition

**Traversing** is a method of control surveying in which a series of connected survey lines (traverse legs) form the framework, and the length and direction of each line are measured using a theodolite (for angles) and a tape/EDM (for distances).

## Types of Traverse

- **Closed Traverse:** Starts and ends at the same point (or at two known points), forming a closed polygon – allows a check on angular and linear closure.
- **Open Traverse (Unclosed):** Does not return to the starting point; used for route surveys (roads, pipelines) where no closure check is directly available.

## Methods of Observing Traverse Angles

- **Method of Included Angles:** Interior (or exterior) angle measured directly at each station.
- **Method of Deflection Angles:** Angle from prolongation of previous line to next line, with R/L direction.
- **Method of Direct / Whole Circle Bearings (Fast Needle Method):** Bearing of each line read directly using the compass box or by carrying forward WCBs.

# Field Procedure for Running a Closed Theodolite Traverse I

1. **Reconnaissance:** Walk the area, select traverse stations ($A, B, C, D, \ldots$) ensuring intervisibility between consecutive stations and reasonably firm, accessible ground.
2. **Reference/Fix First Line's Bearing:** Observe the magnetic bearing (or true bearing via astronomical observation) of the first line for orientation.
3. **At Each Station:** Set up and level the theodolite; measure the included angle (or deflection angle) between the back station and forward station using face left/face right method of repetition.
4. **Measure Line Lengths:** Chain or tape (or EDM) each traverse leg, applying standard corrections (temperature, sag, slope, pull).
5. **Close the Traverse:** Return to the starting station and measure the closing angle; compare the sum of measured interior angles against the theoretical value.

## Theoretical Check for Interior Angles of a Closed Polygon

$$\Sigma\text{Interior Angles} = (2N - 4) \times 90^\circ$$

where $N$ = number of sides of the closed traverse.

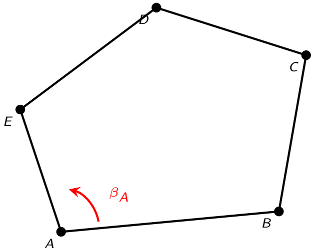



Figure: Closed traverse $ABCDE$: interior angles $\beta_A, \beta_B, \ldots$ and leg lengths measured at each station

# Summary & GTU Exam Focus I

## Key Takeaways

- Errors are classified as instrumental, personal, and natural; most instrumental errors are eliminated by face-change and repetition.
- Theodolite traversing builds a control network of connected lines with measured angles and distances.
- Closed traverse allows a mathematical check ($\Sigma$ interior angles $= (2N - 4) \times 90^\circ$); open traverse does not.
- Angles may be observed as included angles, deflection angles, or whole circle bearings depending on method chosen.

## GTU Exam Important Questions

1. Classify the errors in theodolite surveying with examples of each.
2. How does changing face eliminate instrumental errors? Explain with reasoning.
3. Define theodolite traversing. Differentiate closed and open traverse.
4. State and prove the check for the sum of interior angles of a closed traverse.

# Lecture 07: Agenda & Course Context I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit I:** Theodolite Traverse
**Lecture Topic:** Traverse Computations, Closing Errors, Balancing the Traverse; Gale's Traverse Table & Related Examples.

- **Lecture Objectives:**
  - Compute latitude and departure of each traverse leg from length and bearing.
  - Calculate the total closing error and its direction.
  - Apply Bowditch's Rule to balance (adjust) a closed traverse.
  - Set up and complete a Gale's Traverse Table for a worked numerical example.

## Definitions

For a traverse line of length $L$ and reduced bearing $\theta$ (angle from the N–S meridian):

$$\text{Latitude } (L\cos\theta) = \text{projection of the line on the N–S (meridian) axis}$$
$$\text{Departure } (L\sin\theta) = \text{projection of the line on the E–W axis}$$

## Sign Convention

- Latitude: **Northing (+)** if bearing is in NE or NW quadrant; **Southing (–)** if in SE or SW quadrant.
- Departure: **Easting (+)** if bearing is in NE or SE quadrant; **Westing (–)** if in NW or SW quadrant.

## Computation Sequence

Whole Circle Bearing (WCB) $\rightarrow$ Reduced Bearing (RB, quadrantal) $\rightarrow$ Consecutive Coordinates (Latitude, Departure) $\rightarrow$ Independent (Total) Coordinates of each station.

# Closing Error of a Traverse I

## Why Closing Error Occurs

In a perfectly closed traverse, $\Sigma$Latitude $= 0$ and $\Sigma$Departure $= 0$. In practice, small errors in angle and distance measurement cause these sums to be non-zero.

## Formulas

$$e_L = \Sigma L \quad \text{(algebraic sum of all latitudes, the error in latitude)}$$
$$e_D = \Sigma D \quad \text{(algebraic sum of all departures, the error in departure)}$$
$$\textbf{Total Closing Error } (e) = \sqrt{e_L^2 + e_D^2}$$
$$\textbf{Direction of Closing Error } (\theta) = \tan^{-1}\left(\frac{e_D}{e_L}\right)$$

## Relative Precision (Accuracy Ratio)

$$\text{Relative Precision} = \frac{e}{\text{Perimeter of Traverse}} \quad \text{(expressed as } 1 : n\text{)}$$

Acceptable precision for a GTU-level chain/theodolite traverse is typically 1 : 1000 to 1 : 5000 depending on terrain and instrument grade.

## Principle

Bowditch's Rule (Compass Rule) assumes that errors in angular measurement and linear measurement are of the **same order of magnitude**, so correction is distributed to each line **in proportion to its length**.

## Correction Formulas

$$\text{Correction to Latitude of a line} = -\, e_L \times \frac{\text{Length of that line}}{\text{Perimeter}}$$
$$\text{Correction to Departure of a line} = -\, e_D \times \frac{\text{Length of that line}}{\text{Perimeter}}$$

(The negative sign indicates the correction is applied opposite to the sign of the total error, so that the corrected sums become zero.)

## Transit Rule (Alternative, when angles are more reliable than lengths)

$$\text{Correction to Latitude} = -\, e_L \times \frac{|\text{Latitude of line}|}{\Sigma|\text{Latitudes}|}$$
$$\text{Correction to Departure} = -\, e_D \times \frac{|\text{Departure of line}|}{\Sigma|\text{Departures}|}$$

# Gale's Traverse Table I

## Purpose

Gale's Traverse Table is a standard, systematic tabular format used to record and compute all quantities of a traverse survey in a single organised sheet, from raw field angles to final balanced coordinates.

## Standard Columns of Gale's Table

1. Station and Line
2. Length ($L$)
3. Included Angle (observed)
4. Whole Circle Bearing (WCB)
5. Reduced Bearing (RB)
6. Latitude ($+N/-S$) and Departure ($+E/-W$) – *consecutive (unbalanced) coordinates*
7. Correction to Latitude and Departure (Bowditch)
8. Corrected (Balanced) Latitude and Departure
9. Independent Coordinates (Northing, Easting) of each station

# Gale's Traverse Table II

## Why It Is Used

Keeping all computation steps in one table makes it easy to check the closing error visually and trace any arithmetic mistake back to its source line.

# Worked Numerical Example: Traverse Balancing (Bowditch) I

## Problem

A closed traverse *ABCDA* has the following observed lengths and reduced bearings. Compute the latitude, departure, closing error, and balance the traverse using Bowditch's Rule.

Table: Field Data

| Line | Length (m) | Reduced Bearing |
|---|---|---|
| *AB* | 250.00 | N 30°00′ E |
| *BC* | 300.00 | S 70°00′ E |
| *CD* | 280.00 | S 25°00′ W |
| *DA* | 320.70 | N 64°10′ W |

## Step 1: Compute Latitude and Departure

$$AB: \; L = 250\cos 30^\circ = +216.51, \; D = 250\sin 30^\circ = +125.00$$

$$BC: \; L = -300\cos 70^\circ = -102.61, \; D = +300\sin 70^\circ = +281.91$$

$$CD: \; L = -280\cos 25^\circ = -253.80, \; D = -280\sin 25^\circ = -118.31$$

$$DA: \; L = +320.70\cos 64^\circ 10' = +139.53, \; D = -320.70\sin 64^\circ 10' = -288.72$$

Table: Gale's Traverse Table – Computation and Balancing

| Line | Length | Lat. | Dep. | Corr. Lat. | Corr. Dep. | Bal. Lat. | Bal. Dep. |
|---|---|---|---|---|---|---|---|
| $AB$ | 250.00 | $+216.51$ | $+125.00$ | $+0.08$ | $+0.03$ | $+216.59$ | $+125.03$ |
| $BC$ | 300.00 | $-102.61$ | $+281.91$ | $+0.10$ | $+0.03$ | $-102.51$ | $+281.94$ |
| $CD$ | 280.00 | $-253.80$ | $-118.31$ | $+0.09$ | $+0.03$ | $-253.71$ | $-118.28$ |
| $DA$ | 320.70 | $+139.53$ | $-288.72$ | $+0.10$ | $+0.03$ | $+139.63$ | $-288.69$ |
| $\Sigma$ | 1150.70 | $-0.37$ | $-0.12$ | $+0.37$ | $+0.12$ | 0.00 | 0.00 |

## Step 2: Closing Error, Precision & Bowditch Corrections

$$e_L = -0.37 \text{ m}, \quad e_D = -0.12 \text{ m}$$

$$e = \sqrt{0.37^2 + 0.12^2} = 0.39 \text{ m}, \qquad \text{Relative Precision} = \frac{0.39}{1150.70} \approx \frac{1}{2950}$$

$$\text{Correction to Lat. of } AB = -e_L \times \frac{L_{AB}}{\text{Perimeter}} = 0.37 \times \frac{250.00}{1150.70} = +0.08 \text{ m}$$

A relative precision of about 1:2950 is a reasonable, acceptable closure for a chain-and-theodolite traverse of this length.

## Key Takeaways

- Latitude $= L\cos\theta$, Departure $= L\sin\theta$; signs follow the NE/SE/SW/NW quadrant of the reduced bearing.
- Closing error $e = \sqrt{e_L^2 + e_D^2}$; direction $= \tan^{-1}(e_D/e_L)$; relative precision $= e/\text{perimeter}$.
- Bowditch's Rule distributes correction proportional to line length; Transit Rule distributes proportional to latitude/departure magnitude.
- Gale's Traverse Table organises the full computation – length, bearing, latitude/departure, corrections, balanced values, and coordinates – in one systematic sheet.

## GTU Exam Important Questions

1. Define closing error of a traverse and derive its magnitude and direction.
2. State Bowditch's Rule and derive the correction formula for latitude and departure.
3. Prepare a Gale's Traverse Table for a given set of traverse field data (numerical).
4. Differentiate Bowditch's Rule from the Transit Rule of traverse balancing.

# Lecture 08: Agenda & Unit Transition I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit II:** Trigonometric and Tacheometric Survey (new unit begins here)
**Lecture Topic:** Introduction to Trigonometric Levelling & Methods of Observation.

- **Lecture Objectives:**
  - Define trigonometric levelling and state its purpose.
  - Derive the basic height formula $h = D \tan \theta$ from first principles.
  - Distinguish the case where the base of the object is **accessible** from where it is **inaccessible**.
  - Introduce the **single-plane** (instrument stations in line with the object) and **double-plane** (instrument stations not in line with the object) methods of observation.

## Definition

**Trigonometric levelling** is an indirect method of levelling in which the elevation (RL) of a point is determined from the **observed vertical angle** to that point and the **known (or computed) horizontal distance** to it, using simple trigonometric relations.

## Why It Is Needed

Ordinary (spirit-level) levelling cannot reach the **top** of a tall or inaccessible object – a chimney, water tank, transmission tower, hilltop, or church spire. A theodolite, however, can sight such a point and measure the vertical angle to it.

## Basic Principle (Horizontal Line of Sight as Reference)

If $D$ is the horizontal distance from the instrument to the object, and $\theta$ is the vertical angle of elevation (or depression) observed to the target point, the vertical height $h$ of the target above (or below) the instrument's line of sight is:

$$h = D \tan\theta$$

$$\text{RL of Target} = \text{RL of Instrument Axis} \pm h$$

# Diagram: Basic Principle of Trigonometric Levelling I

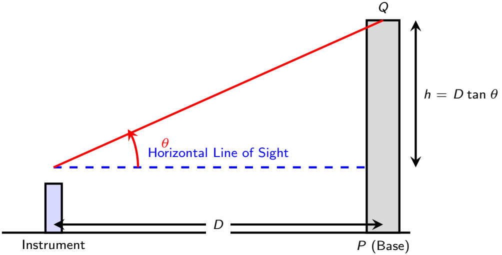



Figure: Height of $Q$ above the horizontal line of sight $= D \tan \theta$

# Classification of Field Conditions I

## Case 1: Base of the Object is Accessible

The horizontal distance $D$ between the instrument station and the base of the object **can be measured directly** with a tape, chain, or EDM (e.g. a chimney standing on open, flat, walkable ground).

## Case 2: Base of the Object is Inaccessible

The base cannot be reached or measured directly (e.g. a tower across a river, on a cliff, or a hilltop) – so $D$ must be found **indirectly** by observing from two instrument stations at a known distance apart.

- **Single-Plane Method:** The two instrument stations and the object are all in the **same vertical plane** (i.e. collinear in plan) – simplest inaccessible case.
- **Double-Plane Method:** The two instrument stations are **not** in the same vertical plane as the object – horizontal angles must also be observed, and the horizontal distance is found using the sine rule before applying $h = D \tan \theta$.

## Preview

Detailed derivations and worked numericals for all these cases are covered in the next two lectures.

# General Field Procedure for Trigonometric Levelling I

1. Set up and level the theodolite at the instrument station; perform temporary adjustment carefully (vertical axis truly vertical is essential for correct vertical angles).
2. Take a backsight on a levelling staff held at a nearby Bench Mark (BM) of known RL; record staff reading $S$. This gives the **Height of Instrument axis**: $\text{HI} = \text{RL of BM} + S$.
3. Sight the target point (top of the object) with face left; measure the vertical angle using the vertical circle verniers (C, D); note whether it is an angle of **elevation** $(+)$ or **depression** $(-)$.
4. Change face (face right) and repeat the vertical angle observation; take the **mean of face-left and face-right readings** to eliminate vertical index error.
5. Compute $D$ (directly measured, or indirectly from a two-station baseline) and apply $h = D \tan\theta$.
6. Compute RL of target $= \text{HI} \pm h$.

## Key Takeaways

- Trigonometric levelling finds elevation from a measured vertical angle and a known/computed horizontal distance: $h = D \tan\theta$.
- Used for inaccessible high points where direct/spirit levelling cannot reach.
- Base-accessible case: distance measured directly.
- Base-inaccessible case: single-plane method (stations in line with object) or double-plane method (stations not in line, needs horizontal angles too).
- Always take the mean of face-left and face-right vertical angle readings.

## GTU Exam Important Questions

1. Define trigonometric levelling and derive the basic height formula $h = D \tan\theta$.
2. Distinguish between base accessible and base inaccessible cases with examples.
3. Explain the difference between the single-plane and double-plane methods of observation.
4. Outline the general field procedure for trigonometric levelling.

# Lecture 09: Agenda & Recap I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit II:** Trigonometric and Tacheometric Survey
**Lecture Topic:** Trigonometric Levelling – Worked Numerical Examples (Base Accessible and Base Inaccessible).

- **Lecture Objectives:**
  - Recall the governing formulas for base-accessible and base-inaccessible (single-plane) trigonometric levelling from Lecture 08.
  - Solve a complete numerical for the **base accessible** case.
  - Solve a complete numerical for the **base inaccessible**, same-vertical-plane case, for both equal and unequal instrument axis levels.

# Recap: Governing Formulas I

## Case 1: Base Accessible

$$h = D \tan\theta, \qquad \text{RL of } Q = \text{RL of BM} + S + h$$

where $D$ is the directly measured horizontal distance to the base, $S$ is the staff reading on the BM, and $\theta$ is the mean (face left/face right) vertical angle.

## Case 2: Base Inaccessible, Same Vertical Plane, Equal Instrument-Axis Level

Two stations $A$ (nearer) and $B$ (farther), $b$ apart, in line with the object; angles $\alpha_1$ at $A$, $\alpha_2$ at $B$:

$$h = \frac{b}{\cot\alpha_2 - \cot\alpha_1}, \qquad D = h\cot\alpha_1$$

## Case 2 (contd.): Unequal Instrument-Axis Levels

If the axis at $A$ is higher than at $B$ by $s = S_B - S_A$:

$$D = \frac{b\tan\alpha_2 - s}{\tan\alpha_1 - \tan\alpha_2}, \qquad h_1 = D\tan\alpha_1$$

## Problem

To determine the RL of the top $Q$ of a water tank, a theodolite was set up at a station $A$, 80 m horizontally from the base of the tank. The angle of elevation to the top of the tank was $32^\circ 15'$ (mean of face left and face right). The staff reading on a BM of RL 215.500 m, taken with the line of sight horizontal, was 1.855 m. Find the RL of the top of the tank.

## Solution.

**Given:** $D = 80$ m, $\alpha = 32^\circ 15' = 32.25^\circ$, RL of BM $= 215.500$ m, $S = 1.855$ m.

**Step 1: Height of Instrument Axis**

$$\text{HI} = \text{RL of BM} + S = 215.500 + 1.855 = 217.355 \text{ m}$$

**Step 2: Height of Tank above Line of Sight**

$$h = D \tan\alpha = 80 \times \tan(32.25^\circ) = 80 \times 0.6307 = 50.456 \text{ m}$$

**Step 3: RL of Top of Tank**

$$\text{RL of } Q = \text{HI} + h = 217.355 + 50.456 = \mathbf{267.811} \text{ m}$$

□

## Problem

Two instrument stations $A$ and $B$, 50 m apart, are set up in line with a transmission tower $Q$ standing on the far bank of a river ($A$ nearer the tower). The angles of elevation to the top $Q$ from $A$ and $B$ are $35^\circ 30'$ and $22^\circ 10'$ respectively. The staff reading on a BM (RL = 180.000 m) is the same from both stations, $S = 1.640$ m. Find the RL of the top of the tower.

## Solution.

**Given:** $b = 50$ m, $\alpha_1 = 35.50^\circ$, $\alpha_2 = 22.1\bar{6}^\circ$, $S_A = S_B = 1.640$ m.
**Step 1: Height $h$**

$$\cot(35^\circ 30') = 1.4019, \qquad \cot(22^\circ 10') = 2.4504$$

$$h = \frac{b}{\cot\alpha_2 - \cot\alpha_1} = \frac{50}{2.4504 - 1.4019} = \frac{50}{1.0485} = 47.686 \text{ m}$$

**Step 2: RL of Top $Q$**

$$\text{RL of } Q = \text{RL of BM} + S + h = 180.000 + 1.640 + 47.686 = \mathbf{229.326} \text{ m}$$

□

# Worked Example 3: Base Inaccessible, Unequal Axis Levels I

## Problem

Stations $A$ and $B$ are 70 m apart, in line with a hill-top station $Q$; $A$ is nearer $Q$. Angles of elevation from $A$ and $B$ are $27^\circ 40'$ and $16^\circ 50'$. Staff reading on BM (RL $= 95.000$ m) from $A$ is 1.415 m and from $B$ is 2.685 m. Determine the RL of $Q$.

## Solution.

**Given:** $b = 70$ m, $\alpha_1 = 27.6\bar{6}^\circ$, $\alpha_2 = 16.8\bar{3}^\circ$, $S_A = 1.415$ m, $S_B = 2.685$ m.
Since $S_B > S_A$, station $A$ is **higher** than $B$ by $s = S_B - S_A = 2.685 - 1.415 = 1.270$ m.
**Step 1: Distance $D$**

$$\tan(27^\circ 40') = 0.5243, \qquad \tan(16^\circ 50') = 0.3026$$

$$D = \frac{b \tan \alpha_2 - s}{\tan \alpha_1 - \tan \alpha_2} = \frac{(70 \times 0.3026) - 1.270}{0.5243 - 0.3026} = \frac{21.182 - 1.270}{0.2217} = 89.816 \text{ m}$$

**Step 2: Height and RL of $Q$**

$$h_1 = D \tan \alpha_1 = 89.816 \times 0.5243 = 47.087 \text{ m}$$

$$\text{RL of } Q = 95.000 + 1.415 + 47.087 = \mathbf{143.502} \text{ m}$$

□

## Key Takeaways

- Base accessible: single station, $D$ measured directly, $h = D\tan\theta$.
- Base inaccessible (same plane, equal axis level): $h = b/(\cot\alpha_2 - \cot\alpha_1)$.
- Base inaccessible (unequal axis level): solve for $D$ first using the tangent-difference formula, then $h_1 = D\tan\alpha_1$.
- Always add the appropriate BM staff reading to get RL, and use mean face-left/face-right vertical angles.

Table: Quick Recap of Formulas Used

| **Case** | **Key Formula** |
|---|---|
| Base Accessible | $h = D\tan\theta$ |
| Inaccessible, Equal Axis | $h = \dfrac{b}{\cot\alpha_2 - \cot\alpha_1}$ |
| Inaccessible, Unequal Axis | $D = \dfrac{b\tan\alpha_2 - s}{\tan\alpha_1 - \tan\alpha_2}$ |

# Summary & GTU Exam Focus II

## GTU Exam Important Questions

1. Solve a numerical to find the RL of an inaccessible point with base accessible (given $D$, $\theta$, BM data).
2. Solve a numerical for base-inaccessible, same-plane, equal-axis-level case.
3. Solve a numerical for base-inaccessible, unequal-axis-level case.

- **Definition:** Trigonometric levelling is an indirect method of levelling in which the relative elevations of various points are determined from the observed vertical angles and horizontal distances.
- **Primary Purpose:** To determine the Reduced Level (RL) of inaccessible points such as mountain peaks, top of chimneys, water tanks, transmission towers, and church spires.
- **Basic Principle:**
  - If horizontal distance $D$ to the object and vertical angle $\alpha$ are known, the height $h$ is given by:
$$h = D \tan \alpha \tag{1}$$
  - Reduced level of target point $Q$:
$$\text{RL of } Q = \text{RL of Instrument Axis} \pm h \tag{2}$$
- **Classification of Field Conditions:**
  1. Base of the object is **accessible**.
  2. Base of the object is **inaccessible** (Instrument stations in the *same vertical plane*).
  3. Base of the object is **inaccessible** (Instrument stations *not in the same vertical plane*).

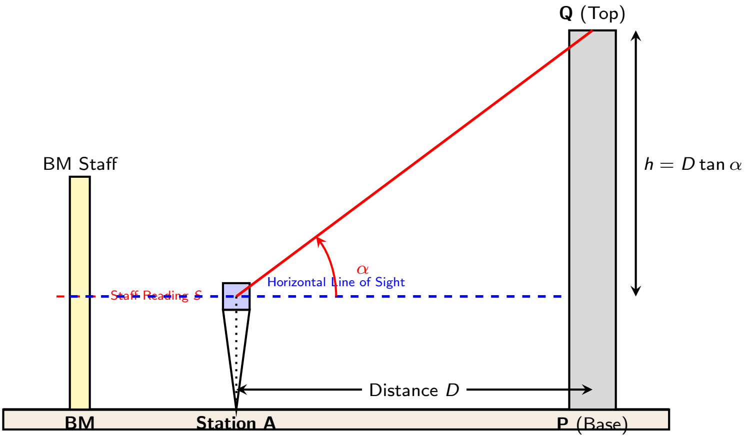
## Field Condition

The horizontal distance $D$ between the instrument station $A$ and the base of the structure $P$ can be measured directly using a tape, chain, or EDM.

- **Procedure:**
  1. Set up the transit theodolite at station $A$ and level it accurately.
  2. Hold the levelling staff on a Benchmark (BM) of known RL and take reading $S$.
  3. Direct the telescope towards target point $Q$ (top of object) and measure vertical angle of elevation $\alpha$.
  4. Measure horizontal distance $D$ from $A$ to base $P$.
- **Formulas:**
  - Height of object above instrument line of sight: $h = D \tan \alpha$
  - Height of Instrument (HI) axis: $\text{HI} = \text{RL of BM} + S$
  - **RL of top point $Q$:**

$$\text{RL of } Q = \text{RL of BM} + S + D \tan \alpha \tag{3}$$

  - *Note for Angle of Depression ($\beta$):* $\text{RL of } Q = \text{RL of BM} + S - D \tan \beta$.


Q (Top)
BM Staff
$h = D \tan \alpha$
$\alpha$
Horizontal Line of Sight
Staff Reading $S$
Distance $D$
BM
Station A
P (Base)

# Case 2: Base Inaccessible – Same Vertical Plane I

## Field Condition

Base $P$ is blocked (e.g. structure across river or on cliff), so distance $D$ cannot be measured directly. Two instrument stations $A$ and $B$ are set up in line with object $Q$.

- **Setup Procedure:**
  1. Select two stations $A$ and $B$ at a known horizontal distance $b$ apart, in line with $Q$.
  2. Measure vertical angle of elevation $\alpha_1$ at $A$ and $\alpha_2$ at $B$ to top point $Q$.
  3. Measure staff readings on BM from both stations ($S_A$ and $S_B$).
- **Three Sub-cases based on Instrument Axis Levels:**
  - **Sub-case 2A:** Instrument axis at $A$ and $B$ at **Same Level** ($S_A = S_B$).
  - **Sub-case 2B:** Instrument axis at $A$ **Higher** than at $B$ ($S_A < S_B$).
  - **Sub-case 2C:** Instrument axis at $A$ **Lower** than at $B$ ($S_A > S_B$).

## Case 2A: Same Instrument Axis Level ($S_A = S_B$) I

- Let $D =$ distance from nearer station $A$ to base $P$.
- Let $b =$ distance between stations $A$ and $B$.
- From $\Delta QA'P'$:

$$h = D\tan\alpha_1 \implies D = \frac{h}{\tan\alpha_1} = h\cot\alpha_1 \quad (4)$$

- From $\Delta QB'P'$:

$$h = (D+b)\tan\alpha_2 \implies D + b = \frac{h}{\tan\alpha_2} = h\cot\alpha_2 \quad (5)$$

- Subtracting the equations:

$$b = h(\cot\alpha_2 - \cot\alpha_1) \implies \mathbf{h} = \frac{\mathbf{b}}{\cot\boldsymbol{\alpha_2} - \cot\boldsymbol{\alpha_1}} \quad (6)$$

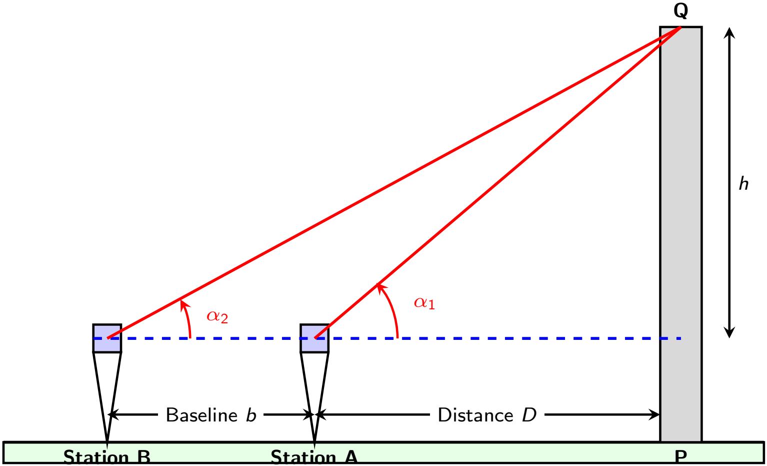
- Alternatively, in terms of sine functions:

$$\mathbf{D} = \frac{\mathbf{b}\sin\boldsymbol{\alpha_2}}{\sin(\boldsymbol{\alpha_1} - \boldsymbol{\alpha_2})}\cos\boldsymbol{\alpha_1}, \quad \mathbf{h} = \frac{\mathbf{b}\sin\boldsymbol{\alpha_1}\sin\boldsymbol{\alpha_2}}{\sin(\boldsymbol{\alpha_1} - \boldsymbol{\alpha_2})} \quad (7)$$

- **RL of Top Point $Q$:**

$$\text{RL of } Q = \text{RL of BM} + S_A + h \quad (8)$$

# TikZ Diagram: Same Instrument Axis Level I



- Let staff reading on BM from $A$ be $S_A$ and from $B$ be $S_B$.
- Difference in instrument axis levels: $s = S_B - S_A$ (Station $A$ is higher by $s$).
- Height of $Q$ above axis at $A$ is $h_1 = D\tan\alpha_1$.
- Height of $Q$ above axis at $B$ is $h_2 = (D + b)\tan\alpha_2$.
- Geometry relation: $h_1 + s = h_2$

$$D\tan\alpha_1 + s = (D + b)\tan\alpha_2 \tag{9}$$

$$D(\tan\alpha_1 - \tan\alpha_2) = b\tan\alpha_2 - s \tag{10}$$

$$\mathbf{D} = \frac{\mathbf{b}\tan\boldsymbol{\alpha_2} - \mathbf{s}}{\tan\boldsymbol{\alpha_1} - \tan\boldsymbol{\alpha_2}} \tag{11}$$

- Once $D$ is calculated:

$$h_1 = D\tan\alpha_1 \tag{12}$$

- **RL of Top Point $Q$:**

$$\text{RL of } Q = \text{RL of BM} + S_A + h_1 \tag{13}$$

- Staff reading from $A$ is $S_A$ and from $B$ is $S_B$. Here $S_A > S_B$.
- Difference in instrument axis levels: $s = S_A - S_B$ (Station $A$ is lower by $s$).
- Height relation: $h_1 - s = h_2$

$$D \tan \alpha_1 - s = (D + b) \tan \alpha_2 \tag{14}$$

$$D(\tan \alpha_1 - \tan \alpha_2) = b \tan \alpha_2 + s \tag{15}$$

$$\mathbf{D} = \frac{\mathbf{b} \tan \alpha_2 + \mathbf{s}}{\tan \alpha_1 - \tan \alpha_2} \tag{16}$$

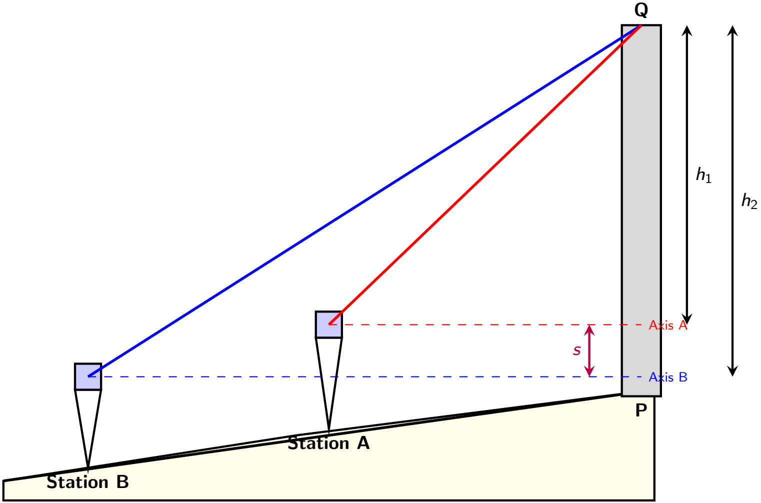
- Once $D$ is obtained:

$$h_1 = D \tan \alpha_1 \tag{17}$$

- **RL of Top Point $Q$:**

$$\text{RL of } Q = \text{RL of BM} + S_A + h_1 \tag{18}$$

# TikZ Diagram: Instrument Axes at Different Levels I



## Field Condition

When obstacles prevent placing instrument stations $A$ and $B$ in line with object $Q$, stations are set up anywhere such that $A$, $B$, and $Q$ form a triangle in horizontal projection.

- **Procedure:**
  1. Measure horizontal baseline length $AB = b$.
  2. Measure horizontal angle $\theta_A = \angle Q'AB$ at station $A$.
  3. Measure horizontal angle $\theta_B = \angle Q'BA$ at station $B$.
  4. Measure vertical angle $\alpha_1$ from $A$ to $Q$, and $\alpha_2$ from $B$ to $Q$.
- **Derivation (Using Sine Rule in $\Delta ABQ'$):**

$$\angle AQ'B = 180^\circ - (\theta_A + \theta_B) \tag{19}$$

$$\mathbf{D_A} = \mathbf{AQ'} = \frac{\mathbf{b}\sin\theta_\mathbf{B}}{\sin(\theta_\mathbf{A} + \theta_\mathbf{B})}, \quad \mathbf{D_B} = \mathbf{BQ'} = \frac{\mathbf{b}\sin\theta_\mathbf{A}}{\sin(\theta_\mathbf{A} + \theta_\mathbf{B})} \tag{20}$$

- **Heights calculation:**

$$h_A = D_A \tan\alpha_1, \quad h_B = D_B \tan\alpha_2 \tag{21}$$

- **RL Calculation & Verification:**

$$\text{RL of } Q(\text{from } A) = \text{RL of BM} + S_A + h_A \tag{22}$$

$$\text{RL of } Q(\text{from } B) = \text{RL of BM} + S_B + h_B \tag{23}$$

*Note: Both calculated RL values should match closely; take average for final value.*

# Curvature and Refraction Corrections I

- When horizontal distance $D$ is large ($D > 2$ km), corrections for earth's curvature and atmospheric refraction must be applied.
- **Curvature Correction ($C_c$):**
  - Earth's surface curves downwards from line of sight.
  - $C_c = \frac{D^2}{2R} = 0.0785D^2$ meters (where $D$ is in km).
  - Always **subtractive** from staff readings / **additive** to vertical heights.
- **Refraction Correction ($C_r$):**
  - Light rays bend downwards due to atmospheric density.
  - $C_r = \frac{1}{7}C_c = 0.0112D^2$ meters (where $D$ is in km).
  - Always **subtractive** from height calculations.
- **Combined Correction ($C$):**

$$C = C_c - C_r = 0.0785D^2 - 0.0112D^2 = \mathbf{0.0673D^2} \text{ meters} \tag{24}$$

- **Corrected Height Formula:**

$$h_{\text{corrected}} = D\tan\alpha + 0.0673D^2 \quad (D \text{ in km, } h \text{ in meters}) \tag{25}$$

## Problem Statement

To find the RL of the top of a chimney $Q$, a theodolite was set up at station $A$ at a distance of 100 m from the base of the chimney. The angle of elevation to the top of the chimney was measured as $24^\circ 30'$. The staff reading on a BM of RL 150.000 m was 1.450 m. Calculate the RL of the top of the chimney.

## Solution.

**Given Data:**

- Horizontal distance $D = 100$ m
- Angle of elevation $\alpha = 24^\circ 30' = 24.5^\circ$
- RL of BM = 150.000 m
- Staff reading on BM $S = 1.450$ m

**Step 1: Calculate Height $h$**

$$h = D \tan \alpha = 100 \times \tan(24.5^\circ) = 100 \times 0.4557 = 45.573 \text{ m} \tag{26}$$

**Step 2: Calculate RL of Top Point $Q$**

$$\text{RL of } Q = \text{RL of BM} + S + h \tag{27}$$

$$= 150.000 + 1.450 + 45.573 = \mathbf{197.023} \text{ m} \tag{28}$$

□

# Numerical Problem 2: Base Inaccessible (Same Axis Level) I

## Problem Statement

An instrument was set up at two stations $A$ and $B$, 60 m apart, in line with a tower $Q$. The angles of elevation from $A$ and $B$ to the top $Q$ were $30^\circ 15'$ and $18^\circ 45'$ respectively. The staff readings on a BM (RL $=$ 200.000 m) from $A$ and $B$ were both equal to 1.500 m. Find the RL of the top of the tower.

## Solution.

**Given Data:**

- Baseline $b = 60$ m
- $\alpha_1 = 30^\circ 15' = 30.25^\circ$, $\alpha_2 = 18^\circ 45' = 18.75^\circ$
- Staff readings $S_A = S_B = 1.500$ m (Same axis level)

**Step 1: Calculate Height $h$**

$$h = \frac{b}{\cot \alpha_2 - \cot \alpha_1} \tag{29}$$

$$\cot(30.25^\circ) = 1.7147, \quad \cot(18.75^\circ) = 2.9459 \tag{30}$$

$$h = \frac{60}{2.9459 - 1.7147} = \frac{60}{1.2312} = \mathbf{48.733} \text{ m} \tag{31}$$

**Step 2: Calculate RL of Top Point $Q$**

$$\text{RL of } Q = 200.000 + 1.500 + 48.733 = \mathbf{250.233} \text{ m} \tag{32}$$

□

## Problem Statement

Two instrument stations $A$ and $B$ are 80 m apart in line with a hill top $Q$. Station $A$ is nearer to the hill. Angles of elevation to $Q$ from $A$ and $B$ are $28^\circ 30'$ and $18^\circ 00'$ respectively. Staff reading on BM (RL = 100.000 m) from $A$ is 1.650 m and from $B$ is 2.850 m. Determine the RL of $Q$.

## Solution.

**Given Data:**

- $b = 80$ m, $\alpha_1 = 28.5^\circ$, $\alpha_2 = 18.0^\circ$
- $S_A = 1.650$ m, $S_B = 2.850$ m
- Station $A$ is HIGHER than $B$ by $s = S_B - S_A = 2.850 - 1.650 = 1.200$ m.

**Step 1: Calculate Distance $D$**

$$D = \frac{b \tan \alpha_2 - s}{\tan \alpha_1 - \tan \alpha_2} \tag{33}$$

$$\tan(28.5^\circ) = 0.54296, \quad \tan(18.0^\circ) = 0.32492 \tag{34}$$

$$D = \frac{(80 \times 0.32492) - 1.200}{0.54296 - 0.32492} = \frac{25.9936 - 1.200}{0.21804} = \mathbf{113.711} \text{ m} \tag{35}$$

**Step 2: Calculate Height $h_1$ and RL of $Q$**

$$h_1 = D \tan \alpha_1 = 113.711 \times 0.54296 = 61.741 \text{ m} \tag{36}$$

$$\text{RL of } Q = 100.000 + 1.650 + 61.741 = \mathbf{163.391} \text{ m} \tag{37}$$

□

Table: Summary of Trigonometric Levelling Formulas

| Field Case | Distance Formula ($D$) | Height Formula ($h$ / $h_1$) |
|---|---|---|
| **1. Base Accessible** | Direct measurement ($D$) | $h = D \tan \alpha$ |
| **2A. Inaccessible (Same Axis)** | $D = \frac{b \cot \alpha_1}{\cot \alpha_2 - \cot \alpha_1}$ | $h = \frac{b}{\cot \alpha_2 - \cot \alpha_1}$ |
| **2B. Inaccessible ($A$ Higher)** | $D = \frac{b \tan \alpha_2 - s}{\tan \alpha_1 - \tan \alpha_2}$ | $h_1 = D \tan \alpha_1$ |
| **2C. Inaccessible ($A$ Lower)** | $D = \frac{b \tan \alpha_2 + s}{\tan \alpha_1 - \tan \alpha_2}$ | $h_1 = D \tan \alpha_1$ |
| **3. Different Vert. Planes** | $D_A = \frac{b \sin \theta_B}{\sin(\theta_A + \theta_B)}$ | $h_A = D_A \tan \alpha_1$ |

## Key Points for GTU Examinations

- Always state assumed variables ($D, b, s, \alpha_1, \alpha_2$) clearly in derivations.
- Draw neat, labeled TikZ-style diagrams in exam answers.
- Pay attention to staff reading differences ($s = S_B - S_A$ vs $s = S_A - S_B$).
- Apply curvature/refraction correction ($+0.0673D^2$ m) when $D > 2$ km.

# Lecture 11: Overview & Learning Objectives I

## Course Details

**Course:** Advance Surveying (DI03006021)
**Unit II:** Trigonometric and Tacheometric Survey
**Target Audience:** GTU Diploma Civil Engineering (Semester IV)

## Learning Objectives

By the end of this lecture, students will be able to:

- Understand the fundamental concept and necessity of **Tacheometric Surveying**.
- Identify the primary **purposes, applications, and advantages** over direct chaining.
- Distinguish the essential constructional features of a **Tacheometer**.
- Comprehend the fundamental **Stadia Principle** using similar isosceles triangles.
- Derive the basic horizontal distance formula: $D = Ks + C$.
- Explain the function of an **Anallatic Lens** and its impact on instrument constants.

# Introduction to Tacheometric Surveying I

## Definition

**Tacheometry** (or Tachemetry / Telemetry) is a branch of angular surveying in which horizontal distances and vertical elevations of points are determined by optical measurements, eliminating the need for direct chaining or taping on the ground.

## Etymology & Origin

- Derived from Greek words:
  - *Takhys* = Swift / Fast
  - *Metron* = Measure
- Hence, it literally means **"Rapid Measurement"**.

## Key Equipment

- **Tacheometer:** A transit theodolite equipped with a stadia diaphragm and an anallatic lens.
- **Stadia Rod / Levelling Staff:** Graduated rod read through the telescope.

# Purpose & Practical Applications I

## Where is Tacheometry Used?

Direct chaining is accurate on smooth, level ground but becomes extremely difficult, tedious, and inaccurate in rough, hilly, or obstructed terrain.

- **Topographic Mapping & Contouring:** Preparing maps of rough, broken, or steep terrain where direct chaining is impractical.
- **Route Location Surveys:** Preliminary location surveys for highways, railways, canals, and pipelines.
- **Hydrographic Surveys:** Measuring distances across rivers, lakes, reservoirs, and marshy areas.
- **Checking Direct Chaining:** Rapidly verifying distances measured with tape or chain to detect gross errors.
- **Reconnaissance Surveys:** Obtaining quick field data where speed is prioritized over extreme precision.

## Advantages

- **High Speed:** Rapid coverage of large areas.
- **Overcomes Obstacles:** Ideal for steep hills, valleys, rivers, and swamps.
- **Simultaneous Data:** Provides horizontal distance and relative elevation ($RL$) in a single observation.
- **Reduced Labor:** Eliminates tape-pulling team in hazardous terrain.

## Limitations

- **Lower Precision:** Less accurate than direct chaining on level ground or modern electronic total stations.
- **Weather Dependent:** Affected by strong wind, fog, refraction, and atmospheric haze.
- **Sight Range Limit:** Staff readings become difficult beyond $100 - 150$ m sight length.

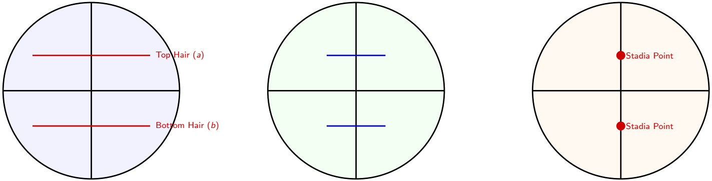
## What distinguishes a Tacheometer from an ordinary Theodolite?

A Tacheometer is a transit theodolite modified with specific optical features:

1. **Stadia Diaphragm:** Fitted with two additional horizontal cross-hairs (stadia lines) placed equidistant above and below the central horizontal cross-hair.
2. **Anallatic Lens:** An additional convex lens fitted inside the telescope between objective and eyepiece to make the additive constant ($C$) zero.
3. **High Magnifying Power:** Telescope magnification of $25\times$ to $30\times$ to read staff graduations clearly at long distances.
4. **Large Aperture:** Objective lens diameter of 35 mm to 45 mm to allow maximum light entry for bright images.
5. **Fixed Constants:** Designed such that Multiplying Constant $K = 100$ and Additive Constant $C = 0$.

## Stadia Diaphragm Patterns

The diaphragm contains three horizontal lines: **Top Stadia Hair**, **Central Cross-hair**, and **Bottom Stadia Hair**.



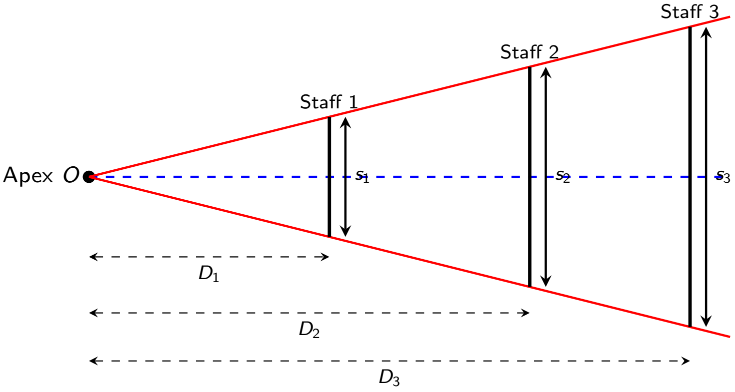

(a) Standard Stadia (b) Short Stadia Hairs (c) Stadia Points

## Staff Intercept ($s$)

Staff Intercept $s$ = Top Hair Reading ($S_1$) − Bottom Hair Reading ($S_3$)

## Principle of Similar Isosceles Triangles

Tacheometry is based on the optical property of **similar isosceles triangles**: the ratio of the distance of the base from the apex to the length of the base is constant.



$$\frac{D_1}{s_1} = \frac{D_2}{s_2} = \frac{D_3}{s_3} = \frac{f}{i} = \text{Constant } K$$

## Optical Geometry for Horizontal Line of Sight

Let:

- $O$ = Optical center of objective lens, $f$ = Focal length of objective.
- $i$ = Stadia hair interval (distance between top and bottom stadia lines).
- $s$ = Staff intercept ($A'B'$ on staff), $c$ = Distance from $O$ to vertical axis of instrument.
- $u$ = Distance of staff from optical center $O$, $v$ = Distance of diaphragm from $O$.
- $D$ = Horizontal distance from instrument vertical axis to staff ($D = u + c$).

## Step 1: Similar Triangles

From conjugate focal relations:

$$\frac{u}{v} = \frac{s}{i} \implies v = \frac{i}{u} \cdot s \quad \text{or} \quad \frac{1}{v} = \frac{s}{i \cdot u}$$

## Step 2: Lens Formula

Applying general lens equation $\frac{1}{f} = \frac{1}{v} + \frac{1}{u}$:

$$\frac{1}{f} = \frac{s}{i \cdot u} + \frac{1}{u} = \frac{1}{u}\left(\frac{s}{i} + 1\right)$$

# Final Distance Equation & Instrument Constants I

## Final Derivation Step

Multiplying both sides by $f \cdot u$:

$$u = \left(\frac{f}{i}\right) s + f$$

Since total horizontal distance $D = u + c$ (where $c$ is axis-to-lens distance):

$$D = \left(\frac{f}{i}\right) s + (f + c)$$

## Standard Tacheometric Distance Formula

$$\mathbf{D = K \cdot s + C}$$

- $\mathbf{K = \frac{f}{i}}$ : **Multiplying Constant** (Stadia Interval Factor). Nominally set to **100**.
- $\mathbf{C = f + c}$ : **Additive Constant** (Instrument Constant). Ranges from 0.3 m to 0.6 m in standard telescopes.

## What is an Anallatic Lens?

An **Anallatic Lens** is an additional convex lens fitted between the objective lens and diaphragm of a tacheometer telescope. Invented by **Porro** in 1823.

## Purpose & Working

- It shifts the apex of the measuring optical triangle exactly to the **vertical axis of rotation** of the instrument.
- This forces the additive constant $C = (f + c) = 0$.
- Reduces distance formula to simple multiplication:

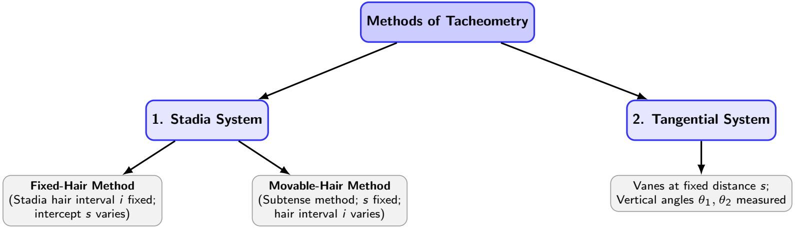
$$\mathbf{D} = \mathbf{100} \cdot \mathbf{s}$$

## Key Advantage

- Eliminates additive constant additions in field calculations.
- Greatly accelerates field book computations.
- Standard feature in modern surveying tacheometers.

## Classification Architecture

Tacheometric surveying methods are broadly divided based on hair movement and observation technique:



# Numerical Example: Horizontal Distance Calculation I

## Problem Statement

A tacheometer equipped with an anallatic lens ($K = 100, C = 0$) was set up at station $A$. The line of sight was horizontal, and staff readings taken on a vertically held staff at station $B$ were:

$$\text{Top Hair } (S_1) = 2.455 \text{ m}, \quad \text{Central Hair } (S_2) = 1.850 \text{ m},$$
$$\text{Bottom Hair } (S_3) = 1.245 \text{ m}$$

Calculate: (i) Staff intercept $s$, (ii) Horizontal distance $AB$, (iii) Verify central hair reading.

# Numerical Example: Horizontal Distance Calculation II

## Solution

1. **Staff Intercept ($s$):**

$$s = S_1 - S_3 = 2.455 - 1.245 = \mathbf{1.210} \text{ m}$$

2. **Horizontal Distance ($D$):**

$$D = K \cdot s + C = 100 \times 1.210 + 0 = \mathbf{121.00} \text{ m}$$

3. **Verification of Central Reading:**

$$S_2 = \frac{S_1 + S_3}{2} = \frac{2.455 + 1.245}{2}$$
$$= 1.850 \text{ m} \quad \text{(Checked!)}$$

## Key Takeaways

- **Tacheometry** measures distances optically without chains/tapes, ideal for rough terrain.
- Uses similar isosceles triangle principles to establish $D = Ks + C$.
- **Multiplying Constant ($K = f/i$):** Usually 100.
- **Additive Constant ($C = f + c$):** Made zero using an **Anallatic Lens**.
- **Fixed-Hair Stadia Method** is the most widely adopted field technique.

## GTU Review Questions

1. Define Tacheometry. Enlist four main purposes of tacheometric surveying.
2. State the fundamental principle of tacheometry with a neat sketch.
3. Derive the expression for horizontal distance $D = Ks + C$ for horizontal line of sight.
4. Explain the necessity and function of an Anallatic Lens in a tacheometer.

# Lecture 12: Overview & Learning Objectives I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit II:** Trigonometric and Tacheometric Survey
**Lecture Topic:** Theory of Stadia Tacheometry, Anallatic Lens, and its Advantages & Disadvantages.

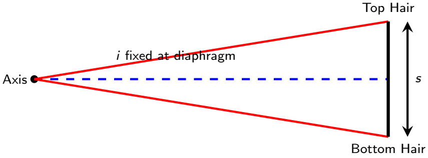
## Recap of Lecture 11

Lecture 11 established the fundamental optical principle of tacheometry and derived $D = Ks + C$ from similar isosceles triangles. This lecture goes deeper into the **fixed-hair stadia method**, the internal **anallatic lens** construction, and a formal comparison with chain surveying.

- **Lecture Objectives:**
  - Explain the working of the fixed-hair stadia method in field practice.
  - Describe the construction and optical positioning of the anallatic lens.
  - Show how the anallatic lens reduces the additive constant $C$ to zero.
  - Compare the advantages and disadvantages of stadia tacheometry against chain surveying.

## Principle

In the **fixed-hair method**, the stadia hair interval $i$ (spacing between the top and bottom stadia hairs on the diaphragm) is permanently *fixed* by the manufacturer. The staff intercept $s$ (difference between top-hair and bottom-hair readings) *varies* in direct proportion to the distance of the staff from the instrument.




## Field Procedure

1. Set up the tacheometer at station $A$ and level it carefully.
2. Hold the graduated staff vertically at the point $B$ whose distance/elevation is required.
3. Sight the staff and read the **Top Hair ($S_1$)**, **Central Hair ($S_2$)**, and **Bottom Hair ($S_3$)**.
4. Compute staff intercept: $s = S_1 - S_3$.
5. Apply the distance formula: $D = Ks + C$.

# Fixed-Hair Stadia Method: Working Principle II

## Why "Fixed-Hair"?

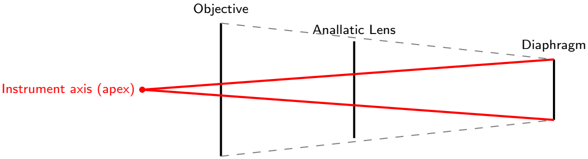
Since $i$ never changes, only $s$ is read in the field for every sighted point – making this method fast and the most widely used stadia technique in practice, as opposed to the movable-hair (subtense) method where $i$ itself must be adjusted and read for every observation.

## The Problem with an Ordinary External-Focusing Telescope

In a plain external-focusing tacheometer, the apex of the measuring triangle lies *inside* the telescope, at a distance $(f + c)$ behind the objective lens. This forces every distance computation to add the constant $C = f + c \approx 0.3$ to 0.6 m, which is tedious and error-prone over many field readings.

## Construction of the Anallatic Lens

An **anallatic lens** is a second, additional convex lens of short focal length $f_2$, permanently fixed *inside* the telescope tube between the objective lens (focal length $f_1$) and the diaphragm, at a fixed distance $d$ from the objective. Invented by **Ignazio Porro** in 1823 (hence also called a *Porro lens*).



## Working Condition

The anallatic lens is positioned such that the distance $d$ between the objective and the anallatic lens satisfies:

$$d = f_1 + f_2$$

This optically shifts the apex of the measuring triangle forward, exactly onto the **vertical axis of the instrument**. Consequently, the effective additive distance $(f + c)$ collapses to zero.

# Effect of Anallatic Lens on Instrument Constants I

## Before vs After the Anallatic Lens

| Feature | Simple External-Focusing | Anallatic (with Porro lens) |
|---|---|---|
| Additive const. $C$ | $0.3 - 0.6$ m | 0 (exactly) |
| Distance formula | $D = Ks + C$ | $D = Ks$ |
| Apex location | Inside telescope | At vertical axis |

## Numerical Illustration

A staff intercept of $s = 1.500$ m is read with $K = 100$.

- **Without** anallatic lens ($C = 0.45$ m): $D = 100(1.500) + 0.45 = \mathbf{150.45}$ m
- **With** anallatic lens ($C = 0$): $D = 100(1.500) = \mathbf{150.00}$ m

Ignoring the additive constant when it is non-zero introduces a systematic error of 0.45 m on every sighting – exactly what the anallatic lens eliminates.

# Advantages & Disadvantages of the Anallatic Lens I

## Advantages

- Reduces field formula to simple $D = Ks$ – faster booking and reduction.
- $C = 0$ remains constant regardless of focusing distance.
- Removes a persistent source of computational (not just observational) error.
- Simplifies training of field staff.

## Disadvantages

- Extra lens absorbs/reflects some light – image slightly less bright.
- Increases manufacturing cost and telescope length.
- Needs precise factory alignment; disturbance requires re-calibration.
- Adds a small amount of additional optical aberration.

# Stadia Tacheometry vs Chain Surveying I

| Aspect | Stadia Tacheometry | Chain Surveying |
|---|---|---|
| Speed | Very fast; one sighting gives $D$ and $RL$ | Slow; distances measured link by link |
| Terrain suitability | Excellent for hilly, broken, marshy ground | Poor on steep/obstructed ground |
| Accuracy | Moderate (1 in 500 to 1000) | High on level ground (1 in 1000+) |
| Elevation data | Obtained simultaneously | Not obtained; needs separate levelling |
| Manpower | One instrument party | Chainmen/followers needed at every link |
| Equipment cost | Higher (tacheometer + staff) | Lower (chain/tape only) |

## Overall Conclusion

Stadia tacheometry trades a small amount of precision for a large gain in speed and terrain versatility, making it the preferred method for reconnaissance, contouring, and route surveys, while chain surveying remains superior for small, level, high-precision plots.

## Key Takeaways

- Fixed-hair stadia method: $i$ is fixed, $s$ varies with distance; $D = Ks + C$.
- Anallatic lens (Porro, 1823): second convex lens placed at $d = f_1 + f_2$ from the objective.
- Its effect: shifts the triangle apex to the instrument axis, making $C = 0$, so $D = Ks$.
- Trade-off: simpler field formula vs. slightly dimmer image and higher cost.
- Stadia tacheometry beats chain surveying in speed and terrain adaptability but loses on raw precision.

## GTU Review Questions

1. Explain the fixed-hair method of stadia tacheometry with the field procedure.
2. Describe the construction and function of an anallatic lens with a neat sketch.
3. Show how the anallatic lens reduces the additive constant to zero.
4. Compare the advantages and disadvantages of tacheometric surveying with chain surveying.

# Lecture 13: Overview & Learning Objectives I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit II:** Trigonometric and Tacheometric Survey
**Lecture Topic:** Methods of Determining the Multiplying Constant ($K$) and Additive Constant ($C$) of a Tacheometer.

- **Lecture Objectives:**
  - Understand why tacheometric constants must be determined/verified for a given instrument.
  - Learn the **field (base line) method** of determining $K$ and $C$ using known distances.
  - Learn the **analytical (instrumental) method** using focal length $f$ and stadia interval $i$.
  - Apply the two-equation technique to compute $K$ and $C$ from staff intercept data.

# Why Determine Tacheometer Constants? I

## Recap: The Distance Formula

$$D = Ks + C$$

where $K$ (multiplying constant) is nominally 100 and $C$ (additive constant) is nominally 0 for an anallatic instrument. However, these are *nominal design values* – every individual instrument must be field-verified.

## When Determination is Necessary

- For a **non-anallatic (simple external-focusing) tacheometer**, $C \neq 0$ and must be found before any distance can be reduced.
- Manufacturing tolerances mean the actual $K$ of a given telescope may differ slightly from the nominal 100.
- After repair, re-collimation, or lens replacement, constants must be re-verified.
- Required periodically as a quality-control / calibration check on office and site instruments.

## Two Broad Approaches

1. **Field Method (Base Line Method):** Purely by field observation on a measured base line – no knowledge of internal optics needed.
2. **Analytical (Instrumental) Method:** By physically/optically determining $f$ and $i$ inside the telescope.

# Field Method: Two Known Distances I

## Procedure

1. Select a fairly level, open piece of ground and set up the tacheometer at one end.
2. Along the line, measure out and drive pegs at two accurately known distances $D_1$ and $D_2$ from the instrument using a chain or tape (independent of the tacheometer).
3. Hold the staff vertical at each peg and take the top-hair and bottom-hair readings; compute staff intercepts $s_1$ and $s_2$.
4. Since the same instrument gives: $D_1 = Ks_1 + C$ and $D_2 = Ks_2 + C$, solve the two simultaneous equations for the two unknowns $K$ and $C$.

## Solving the Simultaneous Equations

Subtracting the two equations eliminates $C$:

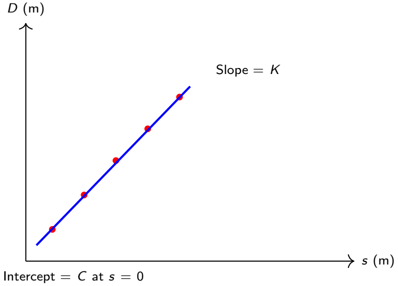
$$D_1 - D_2 = K(s_1 - s_2) \implies K = \frac{D_1 - D_2}{s_1 - s_2}$$

Back-substituting into either equation gives:

$$C = D_1 - Ks_1$$

## Improving Accuracy: Use More Than Two Points

In practice, staff-intercept readings carry small random errors. Using only two pegs makes $K$ and $C$ sensitive to those errors. A more reliable field method uses **5 to 6 pegs** at known distances (e.g., $30, 60, 90, 120, 150$ m), reads the intercept $s$ at each, and fits a straight line $D = Ks + C$ through all the points.



# Field Method: Multiple Pegs & Graphical Solution II

## Interpretation of the Best-Fit Line

- The **slope** of the fitted line gives the multiplying constant $K$.
- The **intercept** on the $D$-axis (value of $D$ where $s = 0$) gives the additive constant $C$.
- This graphical/mean approach averages out small observational errors better than a single pair of points.

# Analytical (Instrumental) Method I

## Principle

Instead of relying only on field measurements, the constants can be computed directly from the telescope's internal optical dimensions, since:

$$K = \frac{f}{i}, \qquad C = f + c$$

## Step 1: Determining Focal Length of Objective ($f$)

- Taken from the manufacturer's specification sheet, **or**
- Computed by focusing the telescope on a well-defined distant object and a near object, measuring the corresponding image travel distance of the objective lens, and applying the thin-lens conjugate relation $\frac{1}{f} = \frac{1}{u} + \frac{1}{v}$.

## Step 2: Determining Stadia Interval ($i$)

- Measured directly on the diaphragm using a microscope fitted with a micrometer eyepiece, **or**
- Derived by sighting a staff held at one precisely known distance $D_0$ and computing $i$ from the corresponding intercept $s_0$: $i = \dfrac{f \cdot s_0}{D_0}$ (with $C$ assumed negligible).

## Step 3: Determining Axis Constant ($c$)

Measured physically as the distance from the instrument's vertical axis to the optical centre of the objective lens when focused for a very distant object; then $C = f + c$.

# Comparison of the Two Methods I

| Aspect | Field (Base Line) Method | Analytical (Instrumental) Method |
|---|---|---|
| Equipment needed | Tape/chain, staff, pegs | Micrometer microscope, optical bench |
| Skill required | Basic field surveying | Specialised optical workshop skill |
| Where performed | On site, quickly | Instrument workshop/laboratory |
| Accounts for real telescope state | Yes (as actually assembled) | Yes, but assumes ideal alignment |
| Common use | Routine site verification | Manufacturing / major recalibration |

## Practical Note

In GTU diploma field practice, the **field (base line) method with two or more known distances** is the standard technique used, since it verifies the instrument exactly as it will be used on site – including any small misalignments that a purely analytical calculation would miss.

## Key Takeaways

- Instrument constants $K$ and $C$ must be verified for every individual tacheometer, especially non-anallatic ones.
- **Field method:** measure staff intercepts at two (or more) precisely known distances, then solve $D = Ks + C$ simultaneously.
- $K = \dfrac{D_1 - D_2}{s_1 - s_2}$, and $C = D_1 - Ks_1$.
- Using multiple pegs and a best-fit line ($D$ vs $s$) improves accuracy over a bare two-point solution.
- **Analytical method:** compute $K = f/i$ and $C = f + c$ from the telescope's internal optical dimensions.

## GTU Review Questions

1. Describe the field method of determining the tacheometric constants $K$ and $C$.
2. Derive the expressions for $K$ and $C$ from two sets of known-distance staff-intercept observations.
3. Explain the analytical method of determining tacheometer constants.
4. Why is a multi-peg graphical method preferred over a two-point calculation in practice?

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit II:** Trigonometric and Tacheometric Survey
**Lecture Topic:** Complete Worked Numerical Example – Determining Tacheometer Constants $K$ and $C$ from Field Staff-Intercept Data.

## Recap of Lecture 13

Lecture 13 introduced the field (base line) method: measure staff intercepts at known distances, then solve $D = Ks + C$ simultaneously for $K$ and $C$. This lecture works through that method completely with real field numbers.

- **Lecture Objectives:**
  - Reduce raw staff readings to staff intercepts.
  - Solve simultaneous equations to obtain $K$ and $C$.
  - Verify the derived constants against an independent check point.
  - Apply the derived constants to find an unknown distance.

# Problem Statement I

## Field Data

A tacheometer was set up at station $O$ on level ground. A staff was held vertically at two pegs $P_1$ and $P_2$ whose horizontal distances from $O$ had been **accurately chained** beforehand. The line of sight was horizontal in both cases. Observations recorded:

| **Peg** | **Chained** $D$ **(m)** | **Top Hair (m)** | **Bottom Hair (m)** |
|---|---|---|---|
| $P_1$ | 100.30 | 1.855 | 0.855 |
| $P_2$ | 200.10 | 2.360 | 0.360 |

**Required:** (i) Staff intercepts $s_1, s_2$; (ii) Multiplying constant $K$; (iii) Additive constant $C$.

# Step 1: Compute the Staff Intercepts I

## Staff Intercept $s$ = Top Hair − Bottom Hair

$$s_1 = 1.855 - 0.855 = \mathbf{1.000} \text{ m} \quad (\text{at } P_1)$$
$$s_2 = 2.360 - 0.360 = \mathbf{2.000} \text{ m} \quad (\text{at } P_2)$$

## Governing Equations

Since $D = Ks + C$ applies to both stations:

$$D_1 = 100.30 = K(1.000) + C \qquad ...(i)$$
$$D_2 = 200.10 = K(2.000) + C \qquad ...(ii)$$

Two equations, two unknowns ($K$ and $C$) – solvable directly.

# Step 2: Solve for $K$ and $C$ I

## Eliminate $C$: Subtract (i) from (ii)

$$200.10 - 100.30 = K(2.000 - 1.000)$$
$$99.80 = K(1.000)$$
$$\boxed{K = 99.80}$$

## Back-substitute into Equation (i)

$$100.30 = 99.80(1.000) + C$$
$$100.30 = 99.80 + C$$
$$\boxed{C = 0.50 \text{ m}}$$

## Result

The tacheometer's actual working constants are $K = 99.80$ and $C = 0.50$ m – close to the nominal $K = 100$, and $C$ within the typical external-focusing telescope range of 0.3 to 0.6 m (this instrument is therefore *not* anallatic).

# Step 3: Verify with an Independent Check Point I

## Third Peg $P_3$ (used only for verification)

A third peg $P_3$ was chained at $D_3 = 150.15$ m. The staff readings there were Top Hair $= 2.108$ m, Bottom Hair $= 0.608$ m.

## Check Computation

$$s_3 = 2.108 - 0.608 = 1.500 \text{ m}$$

$$D_{3,\text{computed}} = Ks_3 + C = 99.80(1.500) + 0.50 = 149.70 + 0.50 = \mathbf{150.20} \text{ m}$$

## Comparison

$$\text{Chained } D_3 = 150.15 \text{ m} \quad \text{vs.} \quad \text{Computed } D_3 = 150.20 \text{ m}$$

Difference $= 0.05$ m over a 150 m sight, i.e. 1 in 3000 – well within acceptable tacheometric accuracy, confirming $K$ and $C$ are reliable.

# Step 4: Apply the Derived Constants I

## Using $K = 99.80$, $C = 0.50$ m for a New Sighting

On the same instrument, a new point $Q$ (unknown distance) is sighted with a horizontal line of sight, giving Top Hair $= 1.960$ m and Bottom Hair $= 1.055$ m.

## Solution

$$
\begin{aligned}
s_Q &= 1.960 - 1.055 = 0.905 \text{ m} \\
D_Q &= Ks_Q + C = 99.80(0.905) + 0.50 \\
D_Q &= 90.319 + 0.50 = \mathbf{90.82} \text{ m}
\end{aligned}
$$

## Why This Matters

Once $K$ and $C$ are established for a specific instrument, *every subsequent field reading* taken with it can be reduced to a horizontal distance using this same pair of constants – there is no need to re-derive them for each new station.

## Key Takeaways

- Staff intercept $s$ = Top Hair − Bottom Hair reading.
- Two known-distance/intercept pairs give two equations in $K$ and $C$; solve by elimination.
- Worked result: $K = 99.80$, $C = 0.50$ m for the example instrument.
- A third, independent check point confirmed the constants to within 0.05 m over 150 m.
- Once determined, $K$ and $C$ are reused for all future readings from that instrument.

## GTU Review Questions

1. Given staff intercepts at two known distances, derive $K$ and $C$ for a tacheometer.
2. Explain how a third check point is used to verify computed tacheometric constants.
3. A tacheometer with $K = 100.2$, $C = 0.35$ m reads a staff intercept of 1.245 m. Find the horizontal distance.

# Lecture 15: Overview & Learning Objectives I

## Course & Unit Information

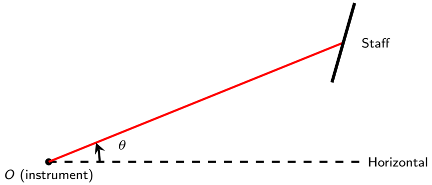
**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit II:** Trigonometric and Tacheometric Survey
**Lecture Topic:** Stadia Tacheometry with Inclined Line of Sight – Staff Held Vertical and Staff Held Normal to Line of Sight.

- **Lecture Objectives:**
  - Understand why an inclined line of sight is unavoidable on sloping ground.
  - Derive the horizontal distance $D$ and vertical (elevation) formula $V$ for a **vertically held staff**.
  - Derive $D$ and $V$ for a staff held **normal to the line of sight**.
  - Learn how to compute the reduced level (RL) of the staff station in each case.

# Why an Inclined Line of Sight? I

## Limitation of the Horizontal-Sight Formula

The simple formula $D = Ks + C$ (Lecture 11) assumes a **horizontal** line of sight. In practice, most terrain tacheometry is done on hilly or undulating ground, where the telescope must be tilted up or down through a vertical angle $\theta$ to sight the staff – so the horizontal formula no longer applies directly.




## Two Standard Field Situations

1. **Staff held vertical** (most common; plumbed as usual).
2. **Staff held normal (perpendicular) to the line of sight** (used on steep gradients, often with a staff-level attachment, to reduce the effect of angular reading error on the intercept).

## Key Idea

The intercept $s$ read on a *vertical* staff is not perpendicular to the inclined line of sight. Resolving it perpendicular to the sight line reduces it by a factor $\cos\theta$; the resulting inclined distance is then further resolved into horizontal and vertical components.

## Derivation

$$\begin{aligned}
\text{Normal intercept} &= s\cos\theta \\
\text{Inclined distance } L &= K(s\cos\theta) + C \\
\text{Horizontal distance } D &= L\cos\theta = Ks\cos^2\theta + C\cos\theta \\
\text{Vertical distance } V &= L\sin\theta = Ks\cos\theta\sin\theta + C\sin\theta = \tfrac{1}{2}Ks\sin 2\theta + C\sin\theta
\end{aligned}$$

## Standard Formulas (Staff Vertical)

$$\boxed{D = Ks\cos^2\theta + C\cos\theta} \qquad \boxed{V = \tfrac{1}{2}Ks\sin 2\theta + C\sin\theta}$$

Sign of $\theta$: positive for angle of **elevation** (staff station above instrument axis), negative for angle of **depression** (staff station below).

# Case B: Staff Held Normal to Line of Sight I

## Key Idea

Here the staff itself is tilted so that it is perpendicular to the line of sight – so the raw intercept $s$ *is* already the normal intercept, and no $\cos\theta$ correction on $s$ is needed. The inclined distance is simply $L = Ks + C$. However, the **central hair reading** $r$ must now be used to shift from the point struck by the line of sight down to the staff foot.

## Standard Formulas (Staff Normal to Line of Sight)

For angle of **elevation** $\theta$:

$$D = L\cos\theta + r\sin\theta \qquad V = L\sin\theta - r\cos\theta$$

For angle of **depression** $\theta$:

$$D = L\cos\theta - r\sin\theta \qquad V = L\sin\theta + r\cos\theta$$

where $L = Ks + C$ and $r$ is the central-hair staff reading.

## Why Use This Method?

On steep slopes, a vertical staff subtends a shortened, foreshortened intercept for a given ground distance; a normal staff always presents its full intercept to the instrument, giving a stronger, less error-prone reading – at the cost of needing a staff-level/clinometer attachment to hold it precisely normal.

# Computing the Reduced Level of the Staff Station I

## General RL Formula

$$RL_{\text{staff station}} = RL_{\text{instrument station}} + h_i + V - (\text{correction if any})$$

where $h_i$ = height of instrument axis (trunnion axis) above the ground at the instrument station.

## Staff Vertical Case

$V$ computed above is the height of the point struck by the **central hair**, not the staff foot. So the central hair reading $r$ must be subtracted:

$$RL_P = RL_O + h_i + V - r$$

## Staff Normal Case

$V$ computed above already accounts for the offset from the sighted point down to the staff foot, so **no further subtraction of $r$ is needed**:

$$RL_P = RL_O + h_i + V$$

| Aspect | Staff Vertical | Staff Normal |
|---|---|---|
| Ease of holding | Simple, uses ordinary plumb | Needs staff-level attachment |
| $D$ formula | $Ks\cos^2\theta + C\cos\theta$ | $L\cos\theta \pm r\sin\theta$ |
| RL correction | Subtract $r$ from $V$ | $r$ already included in $V$ |
| Best suited for | Gentle to moderate slopes | Steep slopes, large $\theta$ |

### Field Practice Note

The staff-vertical method is by far the more commonly used method in routine GTU-level practice because it needs no special staff attachment; the staff-normal method is reserved for steep hill-slope work where accuracy would otherwise suffer.

## Key Takeaways

- Inclined sight is unavoidable on sloping ground; horizontal formula $D = Ks + C$ must be modified.
- **Staff vertical:** $D = Ks\cos^2\theta + C\cos\theta$, $V = \frac{1}{2}Ks\sin 2\theta + C\sin\theta$.
- **Staff normal:** $D = (Ks + C)\cos\theta \pm r\sin\theta$, $V = (Ks + C)\sin\theta \mp r\cos\theta$ (sign per elevation/depression).
- RL of staff station requires subtracting central-hair reading $r$ in the vertical-staff case only.

## GTU Review Questions

1. Derive the horizontal distance and elevation formulas for a vertically held staff with inclined sight.
2. Derive the corresponding formulas for a staff held normal to the line of sight.
3. Why is the central-hair reading $r$ subtracted only in the staff-vertical case when computing RL?
4. Compare the staff-vertical and staff-normal methods with respect to accuracy and field convenience.

## Learning Objectives

- Understand the fundamental principle of the **Fixed Hair Method** (Stadia Method).
- Derive distance and level formulas when line of sight is **horizontal** and staff is vertical.
- Analyze geometry and formulas when line of sight is **inclined** (Angle of Elevation & Depression) with staff vertical.
- Solve GTU-pattern numerical problems step-by-step.

## Core Concepts Covered

- Stadia constants ($k = f/i$, $C = f + d$).
- Horizontal Line of Sight: $D = k \cdot s + C$.
- Inclined Line of Sight: $D = k \cdot s \cdot \cos^2\theta + C \cdot \cos\theta$, $V = \frac{1}{2}k \cdot s \cdot \sin(2\theta) + C \cdot \sin\theta$.

# Principle of Fixed Hair Method I

## What is the Fixed Hair Method?

- The distance between the upper and lower stadia hairs ($i$) is **fixed**.
- The staff intercept ($s$) varies depending on the distance of the staff from the instrument.
- Most widely used method in tacheometric surveying due to speed and simplicity.

## Stadia Constants

- **Multiplying Constant ($k$):**

$$k = \frac{f}{i} \approx 100$$

- **Additive Constant ($C$):**

$$C = f + d \approx 0 \text{ (with Anallactic Lens)}$$

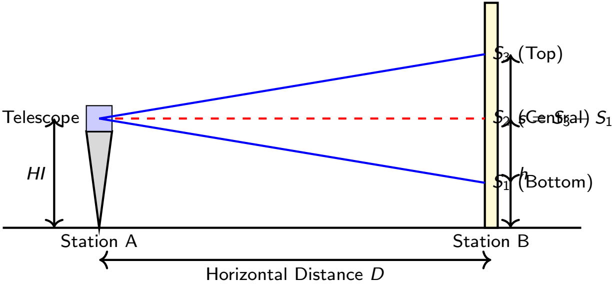
## Key Variables

- $s = S_3 - S_1$: Staff intercept
- $h = S_2$: Central hair reading
- $\theta$: Vertical angle of telescope
- $HI$: Height of Instrument

# Case 1: Horizontal Line of Sight (Staff Vertical) I

## Geometric Setup

Line of sight of telescope is strictly horizontal ($\theta = 0^\circ$), staff held vertically on target point.




## Distance & Height Formulas

$$D = k \cdot s + C = \left(\frac{f}{i}\right) s + (f + d)$$

$$\text{RL of Station B} = \text{RL of Station A} + HI - h$$

## Step-by-Step Procedure for Horizontal Line of Sight

1. Set up instrument at Station A, level accurately, and measure Height of Instrument ($HI$).
2. Hold staff vertically at Station B.
3. Take readings of top ($S_3$), central ($S_2$), and bottom ($S_1$) hairs.
4. Calculate staff intercept: $s = S_3 - S_1$.
5. Verify central hair reading: $S_2 \approx \frac{S_1+S_3}{2}$.
6. Compute distance $D = k \cdot s + C$.
7. Compute RL: $\text{RL}_B = \text{RL}_A + HI - h$ (where $h = S_2$).

## Alternative via Benchmark (BM)

If Backsight ($BS$) is taken on a known Benchmark ($BM$):

$$\text{Height of Instrument (Line of Sight RL)} = \text{RL}_{\text{BM}} + BS$$

$$\text{RL of Station B} = (\text{RL}_{\text{BM}} + BS) - h$$

# Case 2: Inclined Line of Sight (Staff Vertical) I

## Why Inclined Line of Sight?

In hilly or undulating terrain, a horizontal line of sight may pass over the top of the staff or hit the ground. Thus, the telescope is inclined at an angle $\theta$.

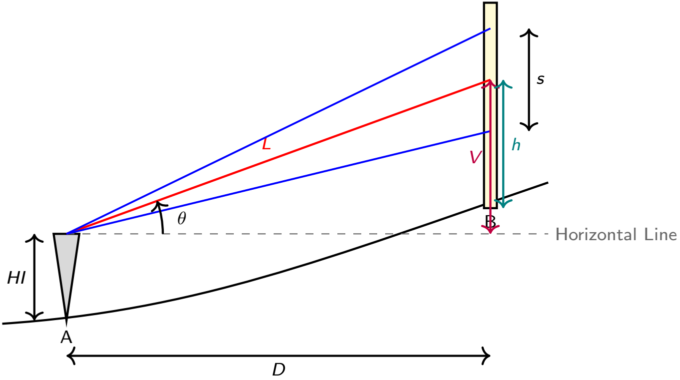
## Angle of Elevation $(+\theta)$

- Telescope pointed **upward**.
- Line of sight elevated above horizontal.
- Target point is higher than instrument height.
- $V$ is added to instrument height.

## Angle of Depression $(-\theta)$

- Telescope pointed **downward**.
- Line of sight depressed below horizontal.
- Target point is lower than instrument height.
- $V$ is subtracted from instrument height.

# Case 2A: Angle of Elevation (Staff Vertical) I



# Case 2A: Formulas for Angle of Elevation I

## Derivation Highlights

Since the staff is held vertical (not perpendicular to line of sight), the effective intercept normal to line of sight is $s' = s \cdot \cos\theta$.

$$\text{Inclined Distance } L = k \cdot s' + C = k \cdot s \cdot \cos\theta + C$$

$$\text{Horizontal Distance } D = L \cdot \cos\theta = (k \cdot s \cdot \cos\theta + C)\cos\theta$$

$$\mathbf{D} = \mathbf{k} \cdot \mathbf{s} \cdot \cos^{\mathbf{2}}\theta + \mathbf{C} \cdot \cos\theta$$

## Vertical Distance ($V$) & Reduced Level ($RL$)

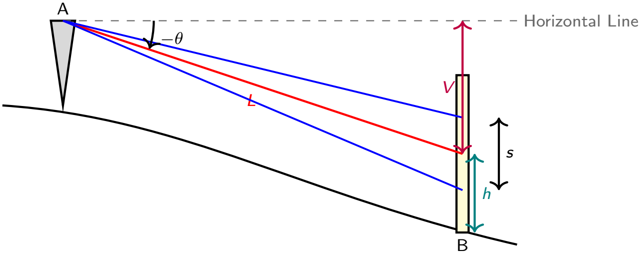
$$V = L \cdot \sin\theta = (k \cdot s \cdot \cos\theta + C)\sin\theta = k \cdot s \cdot \sin\theta\cos\theta + C \cdot \sin\theta$$

$$\mathbf{V} = \frac{\mathbf{1}}{\mathbf{2}}\mathbf{k} \cdot \mathbf{s} \cdot \sin(\mathbf{2}\theta) + \mathbf{C} \cdot \sin\theta$$

$$\mathrm{RL}_{\mathbf{B}} = \mathrm{RL}_{\mathbf{A}} + \mathbf{HI} + \mathbf{V} - \mathbf{h}$$

## Geometry for Angle of Depression $(-\theta)$

When looking downhill, line of sight slopes downward below the horizontal reference line.




## Formulas for Angle of Depression

$$D = k \cdot s \cdot \cos^2\theta + C \cdot \cos\theta$$

$$V = \frac{1}{2} k \cdot s \cdot \sin(2\theta) + C \cdot \sin\theta$$

$$\mathrm{RL}_{\mathbf{B}} = \mathrm{RL}_{\mathbf{A}} + \mathbf{HI} - \mathbf{V} - \mathbf{h}$$

## GTU Problem Statement

A tacheometer with multiplying constant $k = 100$ and additive constant $C = 0.3$ m was set up at station P ($\text{RL}_P = 125.500$ m, $HI = 1.420$ m). Staff was held vertical at Q. Line of sight was horizontal. Staff readings were 1.100 m, 1.800 m, and 2.500 m. Calculate: (i) Horizontal distance PQ, (ii) RL of station Q.

## Solution

1. **Identify Data:** $k = 100$, $C = 0.3$ m, $\text{RL}_P = 125.500$ m, $HI = 1.420$ m. $S_1 = 1.100$ m, $S_2 = 1.800$ m (central $h$), $S_3 = 2.500$ m.
2. **Staff Intercept ($s$):**

$$s = S_3 - S_1 = 2.500 - 1.100 = 1.400 \text{ m}$$

3. **Horizontal Distance ($D$):**

$$D = k \cdot s + C = (100 \times 1.400) + 0.3 = 140.0 + 0.3 = \mathbf{140.300} \text{ m}$$

4. **RL of Station Q:**

$$\text{RL}_Q = \text{RL}_P + HI - h = 125.500 + 1.420 - 1.800 = \mathbf{125.120} \text{ m}$$

# Numerical Example 2: Angle of Elevation I

## GTU Problem Statement

A tacheometer was set up at station A ($RL_A = 210.000$ m, $HI = 1.500$ m). The staff was held vertical at station B. The vertical angle was $+5°30'$. Stadia hair readings were 0.950 m, 1.750 m, and 2.550 m. Take $k = 100$ and $C = 0$. Calculate horizontal distance AB and RL of B.

### Solution

1. **Given Data:** $s = 2.550 - 0.950 = 1.600$ m, $h = 1.750$ m, $\theta = 5^\circ 30' = 5.5^\circ$.
2. **Horizontal Distance ($D$):**

$$D = k \cdot s \cdot \cos^2\theta + C \cdot \cos\theta = 100 \times 1.600 \times \cos^2(5.5^\circ)$$

$$\cos(5.5^\circ) = 0.9954 \implies \cos^2(5.5^\circ) = 0.9908$$

$$D = 160 \times 0.9908 = \mathbf{158.53} \text{ m}$$

3. **Vertical Distance ($V$):**

$$V = \frac{1}{2}k \cdot s \cdot \sin(2\theta) = \frac{1}{2}(100)(1.600)\sin(11^\circ) = 80 \times 0.1908 = \mathbf{15.26} \text{ m}$$

4. **RL of Station B:**

$$\text{RL}_B = \text{RL}_A + HI + V - h = 210.000 + 1.500 + 15.26 - 1.750 = \mathbf{225.010} \text{ m}$$

## GTU Problem Statement

The following observations were taken with a tacheometer ($k = 100, C = 0.2$ m) from station X ($\text{RL}_X = 180.400$ m, $HI = 1.450$ m) to station Y: Vertical angle $= -4°00'$, Staff readings $= 0.800, 1.650, 2.500$ m. Calculate horizontal distance XY and RL of Y.

## Solution

1. **Given Data:** $s = 2.500 - 0.800 = 1.700$ m, $h = 1.650$ m, $\theta = 4.0^\circ$.
2. **Horizontal Distance ($D$):**

$$D = k \cdot s \cdot \cos^2\theta + C \cdot \cos\theta = 100(1.700)\cos^2(4^\circ) + 0.2\cos(4^\circ)$$

$$\cos(4^\circ) = 0.9976, \quad \cos^2(4^\circ) = 0.9951$$

$$D = 170 \times 0.9951 + 0.2 \times 0.9976 = 169.17 + 0.20 = \mathbf{169.37} \text{ m}$$

3. **Vertical Distance ($V$):**

$$V = \frac{1}{2}k \cdot s \cdot \sin(2\theta) + C \cdot \sin\theta = \frac{1}{2}(170)\sin(8^\circ) + 0.2\sin(4^\circ)$$

$$V = 85 \times 0.1392 + 0.2 \times 0.0698 = 11.83 + 0.01 = \mathbf{11.84} \text{ m}$$

4. **RL of Station Y:**

$$\text{RL}_Y = \text{RL}_X + HI - V - h = 180.400 + 1.450 - 11.84 - 1.650 = \mathbf{168.360} \text{ m}$$

## Formula Summary (Fixed Hair Method - Staff Vertical)

| **Line of Sight** | **Horizontal Dist ($D$)** | **Vert Dist ($V$)** | **RL of Target Station** |
|---|---|---|---|
| **Horizontal** | $k \cdot s + C$ | $0$ | $\text{RL}_{\text{inst}} + HI - h$ |
| **Elevation ($+\theta$)** | $k \cdot s \cdot \cos^2\theta + C\cos\theta$ | $\frac{1}{2}k \cdot s \cdot \sin 2\theta + C\sin\theta$ | $\text{RL}_{\text{inst}} + HI + V - h$ |
| **Depression ($-\theta$)** | $k \cdot s \cdot \cos^2\theta + C\cos\theta$ | $\frac{1}{2}k \cdot s \cdot \sin 2\theta + C\sin\theta$ | $\text{RL}_{\text{inst}} + HI - V - h$ |

## Key GTU Exam Tips

- Always check if central hair reading equals average of top and bottom readings: $S_2 = (S_1 + S_3)/2$.
- Ensure vertical angle $\theta$ is converted accurately to decimal degrees before trigonometric calculations.
- Remember: For Anallactic Lens, $C = 0$ and $k = 100$.

# Lecture 17: Overview & Learning Objectives I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit II:** Trigonometric and Tacheometric Survey
**Lecture Topic:** Worked Examples Applying the Inclined Stadia Formulas (Staff Vertical and Staff Normal Cases).

## Recap of Lecture 15

Lecture 15 derived $D = Ks\cos^2\theta + C\cos\theta$ and $V = \frac{1}{2}Ks\sin 2\theta + C\sin\theta$ for a vertical staff, and $D = L\cos\theta \pm r\sin\theta$, $V = L\sin\theta \mp r\cos\theta$ for a staff normal to the line of sight. This lecture applies both to complete numerical problems.

- **Lecture Objectives:**
  - Solve a complete numerical example for a vertical staff with angle of elevation.
  - Solve a complete numerical example for a staff normal to line of sight with angle of depression.
  - Correctly compute the reduced level (RL) of the staff station in each case.

## Problem Statement

A tacheometer fitted with an anallatic lens ($K = 100$, $C = 0$) is set up at station $O$, $RL_O = 100.000$ m, with height of instrument axis $h_i = 1.450$ m. Sighting a **vertically held staff** at point $P$ with an angle of elevation $\theta = 8^\circ 24'$, the readings were:

$$\text{Top Hair} = 2.955 \text{ m}, \quad \text{Central Hair } (r) = 2.205 \text{ m}, \quad \text{Bottom Hair} = 1.455 \text{ m}$$

Find: (i) staff intercept $s$; (ii) horizontal distance $D$; (iii) vertical distance $V$; (iv) $RL_P$.

# Example 1: Solution I

## Step 1: Staff Intercept

$$s = 2.955 - 1.455 = \mathbf{1.500} \text{ m}$$

## Step 2: Horizontal Distance

$$D = Ks\cos^2\theta + C\cos\theta = 100(1.500)\cos^2(8^\circ 24') + 0$$
$$\cos(8^\circ 24') = 0.9893 \implies \cos^2(8^\circ 24') = 0.9786$$
$$D = 150 \times 0.9786 = \mathbf{146.80} \text{ m}$$

## Step 3: Vertical Distance

$$V = \frac{1}{2}Ks\sin 2\theta + C\sin\theta = \frac{1}{2}(150)\sin(16^\circ 48') = 75 \times 0.2890 = \mathbf{21.68} \text{ m}$$

## Step 4: Reduced Level of $P$ (staff vertical – subtract $r$)

$$RL_P = RL_O + h_i + V - r = 100.000 + 1.450 + 21.676 - 2.205$$
$$\boxed{RL_P = 120.92 \text{ m}}$$

# Example 2: Staff Normal to Line of Sight, Angle of Depression I

## Problem Statement

The same tacheometer ($K = 100$, $C = 0$) is set up at station $O$, $RL_O = 250.000$ m, $h_i = 1.500$ m. A staff held **normal to the line of sight** is sighted downhill at a point $Q$ at an angle of depression $\theta = 5^\circ 10'$. The staff intercept read was $s = 0.845$ m and the central hair reading was $r = 1.960$ m. Find: (i) inclined distance $L$; (ii) horizontal distance $D$; (iii) vertical distance $V$; (iv) $RL_Q$.

# Example 2: Solution I

## Step 1: Inclined Distance

$$L = Ks + C = 100(0.845) + 0 = \mathbf{84.500} \text{ m}$$

## Step 2 & 3: Horizontal and Vertical Distances (Depression Case)

Using $D = L\cos\theta - r\sin\theta$ and $V = L\sin\theta + r\cos\theta$, with $\cos(5^\circ 10') = 0.9959$, $\sin(5^\circ 10') = 0.0901$:

$$D = 84.500(0.9959) - 1.960(0.0901) = 84.166 - 0.177 = \mathbf{83.99} \text{ m}$$

$$V = 84.500(0.0901) + 1.960(0.9959) = 7.610 + 1.952 = \mathbf{9.56} \text{ m}$$

## Step 4: Reduced Level of $Q$ (staff normal – $r$ already included, station below axis)

$$RL_Q = RL_O + h_i - V = 250.000 + 1.500 - 9.562$$

$$\boxed{RL_Q = 241.94 \text{ m}}$$

# Key Formula Recap I

| Quantity | Staff Vertical | Staff Normal |
| --- | --- | --- |
| Horizontal $D$ | $Ks\cos^2\theta + C\cos\theta$ | $L\cos\theta \pm r\sin\theta$ |
| Vertical $V$ | $\frac{1}{2}Ks\sin 2\theta + C\sin\theta$ | $L\sin\theta \mp r\cos\theta$ |
| $RL$ correction | Subtract $r$ from $V$ | None; $r$ pre-included |

## Sign Convention Reminder

Upper sign applies to angle of **elevation**, lower sign to angle of **depression**, for the staff-normal formulas. For staff-vertical formulas, simply use a signed $\theta$ (positive for elevation, negative for depression).

## Key Takeaways

- Worked Example 1 (staff vertical, elevation $8^\circ 24'$): $D = 146.80$ m, $RL_P = 120.92$ m.
- Worked Example 2 (staff normal, depression $5^\circ 10'$): $D = 83.99$ m, $RL_Q = 241.94$ m.
- Always identify whether the staff was held vertical or normal before selecting the formula set.
- RL correction differs between the two cases – subtract $r$ only for the vertical-staff case.

## GTU Review Questions

1. A tacheometer ($K = 100$, $C = 0.3$) sights a vertical staff at elevation $6^\circ 00'$ with intercept 1.245 m. Find $D$ and $V$.
2. Solve a complete numerical example for a staff normal to line of sight at an angle of depression.
3. Explain why the RL correction differs between the vertical-staff and normal-staff methods.

# Lecture 18: Overview & Learning Objectives I

## Course & Unit Information

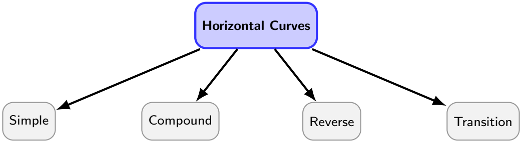
**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit III:** Curves
**Lecture Topic:** Types of Circular Curves; Definitions and Notations used in Curve Surveying.

- **Lecture Objectives:**
  - Understand why curves are introduced in route alignments (roads, railways, canals).
  - Classify circular curves: simple, compound, reverse, and transition.
  - Learn the standard points and terms used to describe a simple circular curve: $PC$, $PT$, $PI$, $R$, $\Delta$, tangent length, length of curve, long chord, mid-ordinate, apex distance.

# Introduction to Curves in Route Surveying I

## Why Are Curves Needed?

Whenever the direction of a road, railway, canal, or pipeline alignment must change, a straight-line intersection cannot be negotiated safely at speed. A **curve** is introduced between the two straight alignments (tangents) to allow a smooth, gradual change of direction.




## Scope of this Lecture

Curves are broadly classed as **horizontal** (changing direction in plan) or **vertical** (changing gradient). This lecture focuses on horizontal circular curves and their four principal types.

# Types of Circular Curves I

## 1. Simple Circular Curve

A single arc of constant radius $R$ connecting two straights, tangential to both. The most common curve used in ordinary road/rail alignment.

## 2. Compound Curve

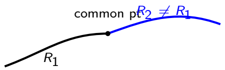
Two or more simple circular curves of *different radii*, curving in the *same direction*, joined at a common tangent point, with their centres on the same side of the alignment. Used where a single radius cannot fit the terrain/right-of-way.

## 3. Reverse Curve

Two simple circular curves (same or different radii) curving in *opposite directions*, joined at a common tangent point – forming an S-shape. Common in railway yards, hill roads, and where the alignment must shift laterally.

## 4. Transition (Easement) Curve

A curve of *continuously varying radius*, introduced between a straight and a circular curve (or between two circular curves) to allow gradual, comfortable development of centrifugal force and superelevation. Covered in detail in Lecture 21.

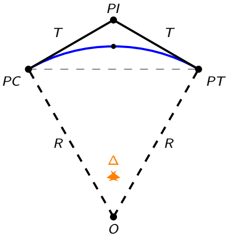



(a) Compound Curve – same direction


(b) Reverse Curve – opposite directions

## Key Points on a Simple Curve

- **Point of Intersection ($PI$)** – also called the *vertex* – where the two straight tangents meet if produced.
- **Point of Curve ($PC$)** – where the curve begins, i.e. where it leaves the back tangent.
- **Point of Tangency ($PT$)** – where the curve ends, i.e. where it joins the forward tangent.
- **Deflection Angle ($\Delta$)** – the angle through which the alignment direction turns, equal to the angle between the tangents at $PI$, also equal to the central angle subtended by the curve at its centre $O$.



# Standard Definitions & Notations I

| Term | Symbol | Meaning / Formula |
|---|---|---|
| Radius | $R$ | Radius of the circular curve |
| Deflection angle | $\Delta$ | Angle between the two tangents |
| Tangent length | $T$ | $PI$ to $PC$ (or $PI$ to $PT$); $T = R\tan(\Delta/2)$ |
| Length of curve | $L_c$ | Arc length $PC$ to $PT$; $L_c = R\Delta\left(\frac{\pi}{180}\right)$ |
| Long chord | $L$ | Straight distance $PC$ to $PT$; $L = 2R\sin(\Delta/2)$ |
| Mid-ordinate | $M_o$ | Midpoint of long chord to midpoint of curve; $M_o = R\left(1 - \cos\frac{\Delta}{2}\right)$ |
| Apex distance | $E$ | $PI$ to midpoint of curve; $E = R\left(\sec\frac{\Delta}{2} - 1\right)$ |

## Note

All these elements will be used directly in Lecture 20 to compute the chainages of $PC$ and $PT$ and to set out the curve in the field.

## Key Takeaways

- Four types of horizontal curves: **simple**, **compound**, **reverse**, **transition**.
- $PC$ = start of curve, $PT$ = end of curve, $PI$ = intersection of the two tangents.
- Deflection angle $\Delta$ equals both the angle between tangents and the central angle at $O$.
- Standard elements: tangent length $T$, length of curve $L_c$, long chord $L$, mid-ordinate $M_o$, apex distance $E$ – all expressed in terms of $R$ and $\Delta$.

## GTU Review Questions

1. Differentiate between simple, compound, and reverse curves with neat sketches.
2. Define $PC$, $PT$, $PI$, and deflection angle $\Delta$ with a labelled diagram.
3. List and define the standard elements of a simple circular curve.
4. Why are transition curves introduced between a straight and a circular curve?

# Lecture Overview & Learning Objectives I

## Unit Context: Simple Circular Curves

In route alignment surveys for roads, railways, and canals, horizontal curves are introduced to achieve smooth changes in direction. Correctly designating these curves and understanding the mathematical relationship between radius and degree of curve is fundamental to engineering design and field staking.

- **Key Topics Covered in this Lecture:**
  - Need for designating a circular curve.
  - Two methods of designation: By Radius ($R$) vs. By Degree of Curve ($D$).
  - **Arc Definition** of degree of curve (Highways preference).
  - **Chord Definition** of degree of curve (Railways preference).
  - Mathematical derivation of $R$-$D$ relation for 20 m and 30 m standard lengths.
  - Exact vs. Approximate formulas for chord definition.
  - Solved GTU numerical examples and conceptual practice questions.
- **Learning Outcome:** Students will be able to convert between radius and degree of curve for any standard chain length, compute errors in approximation, and apply formulas in field survey problems.

- A simple circular curve is defined as a curve of constant radius connecting two straight alignments (tangents).
- **Why Designate Curves?**
  - To define the sharpness or flatness of curvature uniquely on drawings and survey field notes.
  - To calculate linear layout elements (tangent length, length of curve, apex distance, deflection angles).

## 1. Designation by Radius ($R$)

- Curve specified directly by its radius in meters (e.g., $R = 250$ m, 500 m).
- Common in metric countries, urban road design, and structural plans.
- Higher $R \implies$ Flat curve.
- Lower $R \implies$ Sharp curve.

## 2. Designation by Degree ($D$)

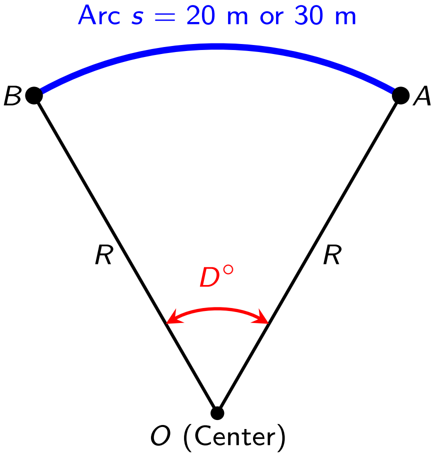
- Curve specified by central angle $D^\circ$ subtended by a standard arc/chord.
- Dominant in field surveying, railway engineering, and curve setting by deflection angles.
- Small $D \implies$ Flat curve.
- Large $D \implies$ Sharp curve.

### Definition (Arc Basis)

The **Degree of Curve ($D$)** under Arc Definition is defined as the central angle (in degrees) subtended at the center of the curve by an **arc of standard length** ($s$).

- **Standard Arc Lengths ($s$):**
  - 20 m arc length (in 20 m metric chain surveying).
  - 30 m arc length (in 30 m metric chain surveying).
  - 100 ft arc length (in FPS / English system).
- **Application:** Widely adopted in **Highway Engineering** because distance along a highway is measured continuously along the curved centerline arc.

# Arc Definition of Degree of Curve II

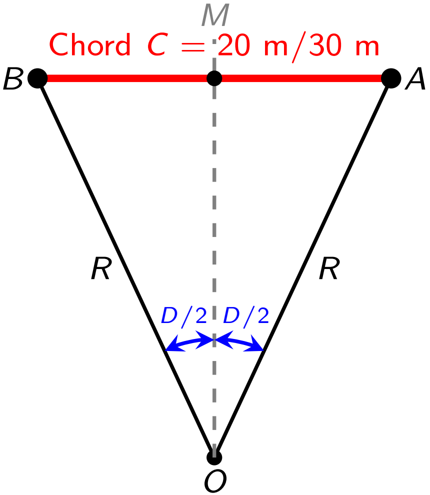


## Definition (Chord Basis)

The **Degree of Curve ($D$)** under Chord Definition is defined as the central angle (in degrees) subtended at the center of the curve by a **chord of standard length** ($C$).

- **Standard Chord Lengths ($C$):**
  - 20 m chord length (for 20 m chain).
  - 30 m chord length (for 30 m chain).
  - 100 ft chord length (in American railway practice).
- **Application:** Extensively used in **Railway Engineering** because railway tracks are pegged and measured using straight chord tape/chain lengths between field stations.

# Chord Definition of Degree of Curve II



# Derivation: Relation between $R$ and $D$ (Arc Definition) I

- Let $R$ = Radius of the curve in meters, and $D$ = Degree of curve in degrees.
- Circumference of a complete circle of radius $R = 2\pi R$.
- Angle subtended by full circumference at the center $= 360^\circ$.
- By simple proportion of arc length $s$ to central angle $D^\circ$:

$$\frac{\text{Arc Length } s}{\text{Circumference}} = \frac{\text{Central Angle } D}{360^\circ} \implies \frac{s}{2\pi R} = \frac{D}{360^\circ}$$

$$D = \frac{360^\circ \cdot s}{2\pi R} = \frac{180 \cdot s}{\pi R} \implies R = \frac{180 \cdot s}{\pi D}$$

## Case 1: For 20 m Chain / Arc ($s = 20$ m)

$$R = \frac{180 \times 20}{\pi D} = \frac{3600}{\pi D} \approx \frac{1145.92}{D} \text{ m}$$

$$\mathbf{R} \approx \frac{\mathbf{1146}}{\mathbf{D}} \text{ meters} \quad \left(\text{or } D \approx \frac{1146}{R}\right)$$

## Case 2: For 30 m Chain / Arc ($s = 30$ m)

$$R = \frac{180 \times 30}{\pi D} = \frac{5400}{\pi D} \approx \frac{1718.87}{D} \text{ m}$$

$$\mathbf{R} \approx \frac{\mathbf{1719}}{\mathbf{D}} \text{ meters} \quad \left(\text{or } D \approx \frac{1719}{R}\right)$$

- Consider isosceles triangle $\triangle OAB$ formed by center $O$ and chord $AB = C$.
- Let $OM \perp AB$. $OM$ bisects chord $AB$ and angle $D$:

$$AM = MB = \frac{C}{2}, \quad \angle AOM = \angle BOM = \frac{D}{2}$$

- In right-angled triangle $\triangle OAM$:

$$\sin\left(\frac{D}{2}\right) = \frac{AM}{OA} = \frac{C/2}{R} = \frac{C}{2R}$$

## Exact Formula for Chord Definition

$$\mathbf{R} = \frac{\mathbf{C}}{\mathbf{2}\sin\left(\frac{\mathbf{D}}{\mathbf{2}}\right)} \quad \text{or} \quad \sin\left(\frac{\mathbf{D}}{\mathbf{2}}\right) = \frac{\mathbf{C}}{\mathbf{2R}}$$

## Approximate Formula for Small Angles

When $D$ is small, $\sin(D/2) \approx \frac{D}{2}$ (in radians) $= \frac{D}{2} \times \frac{\pi}{180^{\circ}} = \frac{\pi D}{360}$. Substituting into exact formula gives $R \approx \frac{C}{2(\pi D/360)} = \frac{180C}{\pi D}$:

- For $C = 20$ m: $\mathbf{R} \approx \frac{\mathbf{1145.92}}{\mathbf{D}} \approx \frac{\mathbf{1146}}{\mathbf{D}}$ m
- For $C = 30$ m: $\mathbf{R} \approx \frac{\mathbf{1718.87}}{\mathbf{D}} \approx \frac{\mathbf{1719}}{\mathbf{D}}$ m

| Parameter | Arc Definition | Chord Definition |
|---|---|---|
| **Base Element** | Standard Arc length ($s$) | Standard Chord length ($C$) |
| **Primary Sector** | Highway Alignment Engineering | Railway Alignment Engineering |
| **Exact Formula** | $R = \frac{180 \cdot s}{\pi D}$ | $R = \frac{C}{2\sin(D/2)}$ |
| **20m Chain Formula** | $R = \frac{1145.92}{D}$ m | $R = \frac{10}{\sin(D/2)}$ m |
| **30m Chain Formula** | $R = \frac{1718.87}{D}$ m | $R = \frac{15}{\sin(D/2)}$ m |
| **Approx. Formula** | Exact (linear relation between $s$ & $R$) | Approx. when $D \leq 5^\circ$ ($R \approx \frac{1146}{D}$ or $\frac{1719}{D}$) |
| **Field Measurement** | Curved baseline measurement | Straight peg-to-peg chord measurement |

Table: Comparison Matrix of Curve Designation Methods

## Problem 1: Radius Calculation from Degree of Curve

Find the radius of a $3^\circ$ circular curve for: (a) 20 m arc length, (b) 30 m arc length.
**Solution:**

a. For 20 m chain arc definition:

$$R = \frac{1145.92}{D} = \frac{1145.92}{3} = \mathbf{381.97} \text{ m} \quad (\text{or } R \approx \frac{1146}{3} = \mathbf{382.00} \text{ m})$$

b. For 30 m chain arc definition:

$$R = \frac{1718.87}{D} = \frac{1718.87}{3} = \mathbf{572.96} \text{ m} \quad (\text{or } R \approx \frac{1719}{3} = \mathbf{573.00} \text{ m})$$

## Problem 2: Degree of Curve from Radius

A highway curve has a radius of 400 m. Determine its degree of curve based on a 20 m chain.
**Solution:**

$$D = \frac{1145.92}{R} = \frac{1145.92}{400} = \mathbf{2.865}^\circ = 2^\circ 51' 53''$$

# Solved Numerical Example 3: Exact vs. Approximate Chord Method I

## Problem 3: Error Analysis in Chord Definition

A 30 m chord is used to establish a sharp railway curve of degree $D = 8^\circ$.

a. Calculate the exact radius of the curve.
b. Calculate the radius using the approximate formula.
c. Find the percentage error involved in the approximation.

**Solution:**

a. **Exact Radius:**

$$R_{\text{exact}} = \frac{C}{2\sin(D/2)} = \frac{30}{2\sin(4^\circ)} = \frac{15}{\sin(4^\circ)} = \frac{15}{0.069756} = \mathbf{215.03} \text{ m}$$

b. **Approximate Radius:**

$$R_{\text{approx}} = \frac{1718.87}{D} = \frac{1718.87}{8} = \mathbf{214.86} \text{ m}$$

c. **Percentage Error:**

$$\text{Error} = R_{\text{exact}} - R_{\text{approx}} = 215.03 - 214.86 = +0.17 \text{ m}$$

$$\% \text{ Error} = \frac{|215.03 - 214.86|}{215.03} \times 100 = \mathbf{0.079\%} \quad \text{(Negligible for field surveys)}$$

## Field Significance of $R$-$D$ Relation

- Allows quick mental and field estimates during alignment reconnaissance.
- Simplifies deflection angle calculation ($\delta = \frac{1718.9 \times c}{R}$ min $= D \times \frac{c}{C}$ deg).
- Ensures standardization across highway (IRC) and railway (Indian Railways) design guidelines.

## GTU Exam Review Questions (DI03006021)

1. Define Degree of Curve. Differentiate between Arc definition and Chord definition with neat sketches. **[4 Marks]**
2. Derive the relation between Radius ($R$) and Degree of Curve ($D$) for a 20 m chain length. **[3 Marks]**
3. Prove that for a 30 m chord length, $R = \frac{15}{\sin(D/2)}$. Calculate $R$ for $D = 4^\circ$. **[4 Marks]**
4. A curve of radius 300 m is to be laid on a road. Find its degree of curve for a 20 m arc. **[2 Marks]**

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit III:** Curves
**Lecture Topic:** Elements of a Simple Circular Curve; Chainage of $PC/PT$; Introduction to Setting Out Methods.

- **Lecture Objectives:**
  - Derive the formulas for tangent length, length of curve, long chord, mid-ordinate, and apex distance.
  - Compute the chainage of $PC$ and $PT$ from the chainage of $PI$.
  - Understand the two broad categories of setting-out methods: linear and instrumental.

## Tangent Length ($T$)

In right triangle $O$-$PC$-$PI$, the angle at $O$ between $OPC$ and $OPI$ is $\Delta/2$ (since $OPI$ bisects $\Delta$), and $OPC \perp PC$-$PI$ (radius is perpendicular to tangent). Hence:

$$\tan\left(\frac{\Delta}{2}\right) = \frac{PC\text{-}PI}{OPC} = \frac{T}{R} \implies \boxed{T = R\tan\left(\frac{\Delta}{2}\right)}$$

## Length of Curve ($L_c$)

The curve is an arc of radius $R$ subtending angle $\Delta$ at the centre. By the standard arc-length relation (angle in degrees):

$$\boxed{L_c = R\,\Delta\left(\frac{\pi}{180}\right)}$$

(If $\Delta$ is already in radians, simply $L_c = R\Delta$.)

## Long Chord ($L$)

Drop perpendicular $OM$ from $O$ to chord $PC$-$PT$; it bisects both the chord and angle $\Delta$:

$$\sin\left(\frac{\Delta}{2}\right) = \frac{L/2}{R} \implies \boxed{L = 2R\sin\left(\frac{\Delta}{2}\right)}$$

## Mid-ordinate ($M_o$)

Distance from the midpoint of the long chord to the midpoint of the curve:

$$M_o = R - R\cos\left(\frac{\Delta}{2}\right) \implies \boxed{M_o = R\left(1 - \cos\frac{\Delta}{2}\right)}$$

## Apex Distance ($E$)

Distance from $PI$ to the midpoint of the curve, along the bisector $O$-$PI$:

$$E = R\sec\left(\frac{\Delta}{2}\right) - R \implies \boxed{E = R\left(\sec\frac{\Delta}{2} - 1\right)}$$

## Formulas

$$\text{Chainage of } PC = \text{Chainage of } PI - T$$
$$\text{Chainage of } PT = \text{Chainage of } PC + L_c$$

## Worked Example

A road curve has deflection angle $\Delta = 48^\circ 30'$, radius $R = 300$ m, and the chainage of $PI$ is 1050.00 m. Find the chainages of $PC$ and $PT$.
**Solution:**

$$T = R\tan(\Delta/2) = 300\tan(24^\circ 15') = 300 \times 0.4505 = 135.15 \text{ m}$$
$$L_c = R\Delta\left(\frac{\pi}{180}\right) = 300 \times 48.5 \times 0.017453 = 253.93 \text{ m}$$
$$\text{Chainage } PC = 1050.00 - 135.15 = \mathbf{914.85} \text{ m}$$
$$\text{Chainage } PT = 914.85 + 253.93 = \mathbf{1168.78} \text{ m}$$

# Setting Out a Simple Curve: Linear Methods I

## When Used

Linear methods use only a tape/chain (no theodolite) and are suitable for **short curves of large radius** on flat, open ground.

## 1. Offsets from the Long Chord

For a point at distance $x$ from the midpoint of the long chord, the perpendicular offset $O_x$ to the curve is:

$$O_x = \sqrt{R^2 - x^2} - \sqrt{R^2 - (L/2)^2}$$

At $x = 0$, this reduces exactly to the mid-ordinate $M_o$.

## 2. Offsets from the Tangent

For a point at distance $x$ measured along the tangent from $PC$, the perpendicular offset $y$ to the curve is:

$$y = R - \sqrt{R^2 - x^2} \quad \left(\text{approximately } y \approx \frac{x^2}{2R} \text{ for small } x\right)$$

# Setting Out a Simple Curve: Instrumental Methods (Introduction) I

## When Used

Instrumental methods use a theodolite (with or without a tape) and give higher accuracy over **long curves and sharp radii**, especially on broken or obstructed ground where a long chord cannot be taped directly.

## Rankine's Method of Tangential (Deflection) Angles – Preview

The theodolite is set up at $PC$, sighted along the tangent (back-sight to $PI$ or the rear straight), and successive points on the curve are located one by one using a small **deflection angle** between the tangent and the chord to each point, combined with the chord length. This is the most widely used instrumental method in field practice and is developed in full, with its formula and worked steps, in **Lecture 21**.

## Other Instrumental Methods (named here, not detailed)

- Two-theodolite method (no chaining required at all).
- Setting out by coordinates using a total station.

## Key Takeaways

- $T = R\tan(\Delta/2)$, $L_c = R\Delta(\pi/180)$, $L = 2R\sin(\Delta/2)$.
- $M_o = R(1-\cos(\Delta/2))$, $E = R(\sec(\Delta/2)-1)$.
- Chainage of $PC$ = Chainage of $PI - T$; Chainage of $PT$ = Chainage of $PC + L_c$.
- Linear setting-out: offsets from long chord or from tangent – suited to short, flat curves.
- Instrumental setting-out (Rankine's deflection angle method, two-theodolite method) suited to long/sharp curves – detailed next lecture.

## GTU Review Questions

1. Derive expressions for tangent length and length of curve of a simple circular curve.
2. A curve has $R = 250$ m, $\Delta = 36^\circ$, chainage of $PI = 2400.00$ m. Find chainages of $PC$ and $PT$.
3. Derive the formula for offsets from the long chord to set out a curve.
4. Distinguish between linear and instrumental methods of setting out a curve.

# Lecture 21: Overview & Learning Objectives I

## Course & Unit Information

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit III:** Curves
**Lecture Topic:** Rankine's Method of Deflection Angles (in depth); Transition Curves – Requirements and Purpose.

## Recap of Lecture 20

Lecture 20 introduced Rankine's method only by name, as the standard instrumental technique for setting out long/sharp curves. This lecture derives its deflection-angle formula fully and works through the field procedure, then explains why transition curves are essential.

- **Lecture Objectives:**
  - Derive the tangential (deflection) angle formula used in Rankine's method.
  - Learn the complete field procedure for setting out a curve by deflection angles.
  - Understand why an abrupt change from straight to circular curve is undesirable.
  - List the requirements of an ideal transition curve.

# Rankine's Method of Deflection Angles: Formula I

## Principle

By the tangent-chord angle theorem, the angle between a tangent to a circle and a chord drawn from the point of tangency equals **half** the central angle subtended by that chord. This angle is called the **deflection (tangential) angle**, $\delta$.

## Derivation

For a chord of length $c$ on a curve of radius $R$, the chord subtends (approximately, for the short chords used in practice) a central angle of $\dfrac{c}{R}$ radians. The deflection angle is half of this:

$$\delta \text{ (radians)} = \frac{c}{2R}$$

Converting to minutes of arc (1 rad $= \frac{180 \times 60}{\pi}$ minutes $= 3437.75'$):

$$\boxed{\delta \text{ (minutes)} = \frac{1718.9\, c}{R}}$$

## Cumulative Deflection Angles

For the $i$-th peg, the **total** angle to be turned off from the initial tangent direction is the running sum of all individual chord deflections up to that point: $\delta_i = \delta_1 + \delta_2 + \cdots + \delta_i$. At $PT$, the cumulative deflection angle must equal $\Delta/2$ – this is the standard field check.

1. Set up the theodolite exactly over $PC$ and level it.
2. Set the horizontal circle to $0^\circ 00'$ and sight the instrument along the back tangent towards $PI$.
3. Compute the chord lengths: a short **sub-chord** from $PC$ to the first round-chainage peg, then **full chords** (usually 20 m or 30 m) between successive pegs, and a final sub-chord into $PT$.
4. Compute the individual deflection angle for each chord using $\delta = 1718.9\, c/R$ (minutes), and accumulate them.
5. Turn the telescope to the cumulative deflection angle $\delta_i$ for peg $i$; simultaneously, an assistant holds a chain/tape of length equal to the chord from the previous peg. The peg is fixed where the sighted line and the chord-length arc intersect.
6. Repeat for each peg in sequence until $PT$ is reached. The final cumulative angle must check out to $\Delta/2$.

# Rankine's Method: Worked Illustration I

## Data (from Lecture 20's Example)

$R = 300$ m, $\Delta = 48^\circ 30'$, Chainage $PC = 914.85$ m, Chainage $PT = 1168.78$ m, normal chord $= 20$ m.

| **Peg (chg.)** | **Chord (m)** | **Indiv.** $\delta$ | **Cumulative** $\delta$ |
|---|---|---|---|
| 920.00 | 5.15 | $0^\circ 29' 31''$ | $0^\circ 29' 31''$ |
| 940.00 | 20.00 | $1^\circ 54' 35''$ | $2^\circ 24' 06''$ |
| 960.00 | 20.00 | $1^\circ 54' 35''$ | $4^\circ 18' 41''$ |
| $\vdots$ | $\vdots$ | $\vdots$ | $\vdots$ |
| $PT$(1168.78) | 8.78 | $0^\circ 50' 18''$ | $\mathbf{24^\circ 15' 00''}$ |

## Check

Cumulative deflection angle at $PT = 24^\circ 15' 00'' = \Delta/2$ exactly – confirming the field work closed correctly.

# Transition Curves: Why Are They Needed? I

## The Problem with a Direct Straight-to-Circular Junction

On a straight, the radius is infinite and centrifugal force on a moving vehicle is zero. The instant the vehicle enters a circular curve of finite radius $R$, centrifugal force appears **suddenly and fully**, causing a jerk, lateral discomfort, risk of skidding, and excessive wear on tyres/rails.

## Purpose of a Transition Curve

- Provides a **gradual introduction of centrifugal force** as radius decreases smoothly from infinity to $R$.
- Allows **superelevation (banking)** to be developed gradually, in step with increasing curvature, instead of applied abruptly.
- Improves **passenger comfort and safety** at the design speed.
- Improves the **visual/aesthetic appearance** of the alignment.
- Reduces wear on vehicle tyres and rail/pavement surfaces caused by sudden lateral shock.

1. It must be **tangential to the straight**: at its starting point, its radius of curvature must be infinite (zero curvature), matching the straight exactly.
2. It must **meet the circular curve tangentially**: at its end point, its radius of curvature must equal exactly $R$, the radius of the circular curve it joins.
3. Its **curvature must increase at a uniform rate** with distance along the curve – i.e. $\frac{1}{\rho}$ (curvature) is directly proportional to the length travelled along the transition. This uniform-rate property is what makes the introduction of centrifugal force and superelevation smooth and comfortable.
4. Its **length** must be sufficient to allow the design rate of gain of radial acceleration and the required superelevation to be attained comfortably at the design speed.

## Common Transition Curve Shapes

Curves satisfying condition 3 (curvature $\propto$ length) include the **clothoid (Euler's spiral)** – the theoretically ideal shape – along with practical approximations such as the **cubic parabola** and **cubic spiral (Bernoulli's lemniscate)**, commonly used in Indian highway and railway practice.

## Key Takeaways

- Rankine's deflection angle: $\delta$ (minutes) $= 1718.9\, c/R$; angles are accumulated peg by peg.
- Total cumulative deflection angle at $PT$ must check to exactly $\Delta/2$.
- Transition curves gradually introduce centrifugal force and superelevation between a straight and a circular curve.
- An ideal transition curve: tangential to the straight at start, tangential to the circular curve (radius $R$) at end, curvature increasing uniformly with length.

## GTU Review Questions

1. Derive the tangential angle formula used in Rankine's method of setting out a curve.
2. Describe the complete field procedure for Rankine's method of deflection angles.
3. Why is a transition curve necessary between a straight and a circular curve? State its purpose.
4. List the requirements that an ideal transition curve must satisfy.

- **Definition of Vertical Curve:**
  - A curve provided in the vertical alignment of a highway or railway to join two intersecting gradients smoothly.
  - Avoids sudden changes in grade, preventing shock and discomfort to passengers.
- **Need for Vertical Curves:**
  - **Passenger Comfort:** Gradual change in gravity forces and vertical acceleration.
  - **Safety & Sight Distance:** Provides adequate stopping sight distance (SSD) and overtaking sight distance (OSD).
  - **Vehicle Operations:** Smooth transitions reduce vehicle wear and power fluctuations.
  - **Drainage & Aesthetics:** Ensures proper slope for water discharge and visually pleasing road profile.

## Difference Between Horizontal and Vertical Curves

| **Parameter** | **Horizontal Curve** | **Vertical Curve** |
|---|---|---|
| Plane of Curve | Horizontal plane $(X - Y)$ | Vertical plane $(X - Z)$ |
| Purpose | Changes direction of alignment (heading) | Changes gradient/slope of alignment |
| Dominant Force | Centrifugal force (countered by super-elevation) | Gravitational force & vertical acceleration |
| Shape Used | Circular arc, transition spiral | Simple parabola $(y = ax^2 + bx + c)$ |

# Types of Vertical Curves: Summit and Sag Curves I

## 1. Summit Curve (Crest Curve)

- Curve is **convex upwards**.
- Formed when:
  - Upgrade meets downgrade ($+g_1$ and $-g_2$).
  - Steeper upgrade meets flatter upgrade ($+g_1 > +g_2$).
  - Flatter downgrade meets steeper downgrade ($-g_1 < -g_2$).
  - Upgrade meets level grade ($+g_1$ and 0%).
- **Design Criterion:** Primary governing factor is **Sight Distance** (SSD, OSD).

## 2. Sag Curve (Valley Curve)

- Curve is **concave upwards**.
- Formed when:
  - Downgrade meets upgrade ($-g_1$ and $+g_2$).
  - Steeper downgrade meets flatter downgrade ($-g_1 > -g_2$).
  - Flatter upgrade meets steeper upgrade ($+g_1 < +g_2$).
  - Downgrade meets level grade ($-g_1$ and 0%).
- **Design Criterion:** Governing factors are **Headlight Sight Distance (HSD)** at night and **Comfort Factor**.

- **Why a Simple Parabola is Preferred?**
  - The rate of change of grade is constant along the curve ($\frac{d^2y}{dx^2}$ = constant).
  - Simple calculations for offsets and elevations at regular stations.
  - Provides smooth acceleration and comfortable driving dynamics.
  - Equal tangent lengths: The VPI bisects the horizontal distance between VPC and VPT ($x_{\text{VPI}} = L/2$).

## General Parabolic Equation

Let the initial tangent point (VPC) be the origin $(0, 0)$.

$$y = ax^2 + bx + c$$

At $x = 0$, $y = 0 \implies c = 0$.
At $x = 0$, slope $\frac{dy}{dx} = g_1 \implies b = g_1$ (where grade $g_1$ is expressed as decimal or fraction).
Rate of change of grade $r = \frac{g_2 - g_1}{L} = 2a \implies a = \frac{g_2 - g_1}{2L} = \frac{-N}{2L}$.

## Key Parabolic Properties

1. Tangent correction (offset) $y$ at distance $x$ from VPC:

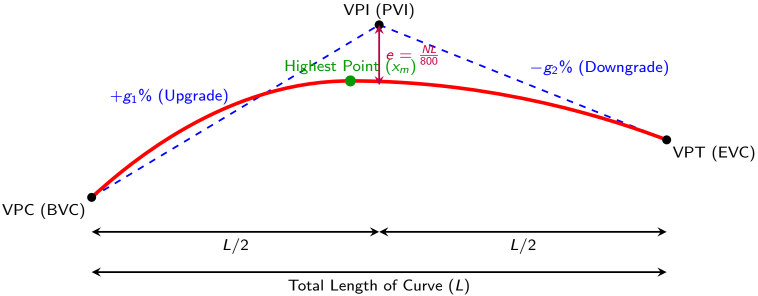
$$y = \frac{Nx^2}{200L} \quad \text{or} \quad y = \frac{rx^2}{2}$$

   where $N = |g_1 - g_2|\%$ is the total algebraic difference in grades.

2. Maximum offset at mid-point ($x = L/2$):

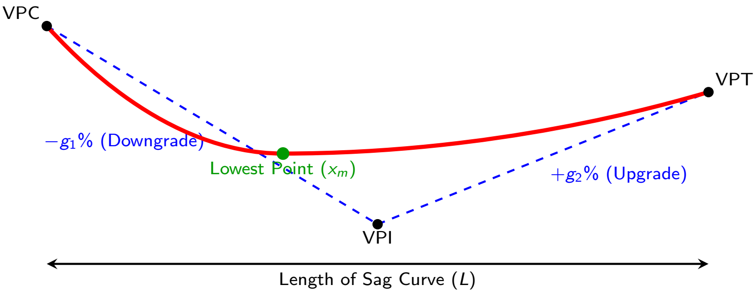
$$y_{\max} = \frac{NL}{800} = \frac{e}{2}$$

**Summit Vertical Curve Geometry**




**Sag Vertical Curve Geometry**

# Schematic Diagrams of Summit and Sag Curves II



## Essential Terminology & Relations

- **VPC (BVC):** Vertical Point of Curvature (Beginning of Vertical Curve).
- **VPI (PVI):** Vertical Point of Intersection.
- **VPT (EVC):** Vertical Point of Tangency (End of Vertical Curve).
- **Algebraic Grade Difference ($N$):**

$$N = g_1 - g_2 \quad \text{(with proper algebraic sign, e.g., } + g_1 - (-g_2) = g_1 + g_2\text{)}$$

- **Chainages (Stationing):**

$$\text{Chainage of VPC} = \text{Chainage of VPI} - \frac{L}{2}$$

$$\text{Chainage of VPT} = \text{Chainage of VPI} + \frac{L}{2}$$

- **Elevations (RL) of Tangent Points:**

$$\text{RL of VPC} = \text{RL of VPI} - g_1 \times \left(\frac{L}{2}\right)$$

$$\text{RL of VPT} = \text{RL of VPI} + g_2 \times \left(\frac{L}{2}\right)$$

## Location of Highest or Lowest Point

Setting derivative $\frac{dy_{\text{curve}}}{dx} = 0$:

$$x_m = \frac{g_1 L}{g_1 - g_2} = \frac{g_1 L}{N} \quad \text{(measured from VPC)}$$

- High point occurs on Summit curve only if gradients are of opposite signs ($+g_1$ and $-g_2$).
- Low point occurs on Sag curve only if gradients are of opposite signs ($-g_1$ and $+g_2$).

1. **Determine Algebraic Change of Grade ($N$):** Calculate $N = g_1 - g_2$ (keep track of signs).
2. **Compute Length of Vertical Curve ($L$):** Based on design speed, sight distance (SSD/OSD), or GTU problem statement.
3. **Calculate Stationing and Tangent Elevations:** Find Chainages and RLs of VPC, VPI, and VPT.
4. **Compute Tangent Offsets ($y_x$):** For any station at distance $x$ from VPC:

$$y_x = \frac{Nx^2}{200L}$$

5. **Determine Curve Elevations (RL of Curve):**
   - **For Summit Curve:** RL of Curve = Tangent RL $- y_x$
   - **For Sag Curve:** RL of Curve = Tangent RL $+ y_x$
6. **Locate High/Low Point:** Find $x_m = \frac{g_1 L}{N}$ and compute its offset and RL.

# Design Criteria for Length of Vertical Curves I

## Summit Curve Length based on Sight Distance ($S$)

Let $h_1$ = driver's eye height (1.2 m), $h_2$ = object height (0.15 m for SSD, 1.2 m for OSD).

- **Case 1: When Length $L > S$ (Curve longer than Sight Distance):**

$$L = \frac{NS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2} \implies L_{\text{SSD}} = \frac{NS^2}{4.4}$$

- **Case 2: When Length $L < S$ (Curve shorter than Sight Distance):**

$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{N} \implies L_{\text{SSD}} = 2S - \frac{440}{N}$$

## Sag Curve Length Criteria

- **Headlight Sight Distance (HSD):** Headlight height $h_1 = 0.75$ m, beam tilt angle $\alpha = 1^\circ$.

$$L = \frac{NS^2}{1.5 + 0.035S} \quad (L > S)$$

- **Comfort Criterion (Vertical Acceleration $C \leq 0.6$ m/s$^3$):**

$$L = 2\left(\frac{v^3 N}{C}\right)^{1/2}$$

## Problem Statement

An upgrade gradient of $+2.0\%$ meets a downgrade gradient of $-1.5\%$ at a VPI located at chainage 1000 m with an elevation of 150.000 m. The required length of vertical curve is 200 m.
**Calculate:**

(a) Chainage and RL of VPC and VPT.

(b) Location and RL of the highest point on the curve.

(c) RL of curve points at 40 m intervals.

# Solved Example 1: Summit Vertical Curve II

## Solution Steps

**1. Basic Parameters:**

- $g_1 = +2.0\%$, $g_2 = -1.5\%$, $L = 200$ m.
- Algebraic Grade Change $N = g_1 - g_2 = +2.0 - (-1.5) = 3.5\%$.

**2. Stationing and Tangent RLs:**

- Chainage of VPC $= 1000 - \frac{200}{2} = 900$ m.
- Chainage of VPT $= 1000 + \frac{200}{2} = 1100$ m.
- RL of VPC $=$ RL of VPI $- g_1 \times (L/2) = 150.000 - \frac{2.0}{100} \times 100 = 148.000$ m.
- RL of VPT $=$ RL of VPI $+ g_2 \times (L/2) = 150.000 + \frac{-1.5}{100} \times 100 = 148.500$ m.

**3. Location of Highest Point:**

$$x_m = \frac{g_1 L}{N} = \frac{2.0 \times 200}{3.5} = 114.286 \text{ m from VPC}$$

- Chainage of highest point $= 900 + 114.286 = 1014.286$ m.
- Tangent RL at highest point $= 148.000 + \frac{2.0}{100} \times 114.286 = 150.286$ m.
- Offset $y_m = \frac{N x_m^2}{200L} = \frac{3.5 \times (114.286)^2}{200 \times 200} = 1.143$ m.
- RL of Highest Point $= 150.286 - 1.143 = 149.143$ m.

# Tabulated Computation for Example 1 I

## Curve Offset Formula

$y = \frac{Nx^2}{200L} = \frac{3.5x^2}{200 \times 200} = \frac{3.5x^2}{40000} = 0.0000875\,x^2$

| Point | Chainage (m) | Dist $x$ from VPC (m) | Tangent RL (m) | Offset $y$ (m) | Curve RL (m) |
|---|---|---|---|---|---|
| VPC | 900 | 0 | 148.000 | 0.000 | 148.000 |
| 1 | 940 | 40 | 148.800 | 0.140 | 148.660 |
| 2 | 980 | 80 | 149.600 | 0.560 | 149.040 |
| VPI | 1000 | 100 | 150.000 | 0.875 | 149.125 |
| Max Pt | 1014.286 | 114.286 | 150.286 | 1.143 | 149.143 |
| 3 | 1020 | 120 | 150.400 | 1.260 | 149.140 |
| 4 | 1060 | 160 | 151.200 | 2.240 | 148.960 |
| VPT | 1100 | 200 | 152.000* | 3.500 | 148.500 |

Table: Vertical Curve Station Elevations (*Note: Tangent extended from $g_1$ upgrade)

## Calculation Check

Check Curve RL at VPT:

$$\text{Curve RL at VPT} = \text{Extended } g_1 \text{ Tangent RL} - \text{Total Offset } y_{\text{end}}$$

Extended Tangent RL $= 148.000 + 0.02 \times 200 = 152.000$ m.

$y_{\text{end}} = \frac{3.5 \times (200)^2}{40000} = 3.500$ m.

Curve RL at VPT $= 152.000 - 3.500 = 148.500$ m ✓ (Matches VPT RL calculated from $g_2$!).

## Problem Statement

A downgrade of $-3.0\%$ meets an upgrade of $+2.0\%$ at a VPI chainage of 1500 m with RL 210.500 m. The length of the sag curve is 160 m.
**Determine:**

(a) Stationing and RL of VPC and VPT.

(b) Distance and RL of the lowest point on the curve.

(c) Curve RLs at 40 m intervals.

# Solved Example 2: Sag (Valley) Vertical Curve II

## Solution

1. **Grade Difference & Geometry:**
   - $g_1 = -3.0\%$, $g_2 = +2.0\%$, $L = 160$ m.
   - $N = g_1 - g_2 = -3.0 - (+2.0) = -5.0\% \implies |N| = 5.0\%$.
2. **Stationing and Tangent RLs:**
   - Chainage of VPC $= 1500 - 80 = 1420$ m.
   - Chainage of VPT $= 1500 + 80 = 1580$ m.
   - RL of VPC $= 210.500 + \frac{3.0}{100} \times 80 = 212.900$ m.
   - RL of VPT $= 210.500 + \frac{2.0}{100} \times 80 = 212.100$ m.
3. **Lowest Point Location:**

$$x_m = \frac{|g_1|L}{|N|} = \frac{3.0 \times 160}{5.0} = 96.000 \text{ m from VPC}$$

   - Chainage of lowest point $= 1420 + 96 = 1516$ m.
   - Tangent RL at $x = 96$ m $= 212.900 - 0.03 \times 96 = 210.020$ m.
   - Offset $y_m = \frac{5.0 \times (96)^2}{200 \times 160} = 1.440$ m.
   - **RL of Lowest Point (Sag Curve $\implies$ Add Offset):**

$$\text{RL} = 210.020 + 1.440 = 211.460 \text{ m.}$$

# Solved Example 3: Related Curve Calculations (Circular Curve Review) I

## Comprehensive GTU Exam Problem

Two straights intersect at chainage 1250.00 m with a deflection angle $\Delta = 45^\circ 00'$. A simple circular curve of radius $R = 300$ m is to be set out.

**Calculate all fundamental elements of the curve:**

(i) Tangent Length ($T$)

(ii) Curve Length ($L$)

(iii) Chainages of $T_1$ (PC) and $T_2$ (PT)

(iv) Long Chord ($C$)

(v) Mid-ordinate ($M$) and Apex Distance ($E$)

# Solved Example 3: Related Curve Calculations (Circular Curve Review) II

## Step-by-Step Solution

1. **Tangent Length ($T$):**

$$T = R\tan\left(\frac{\Delta}{2}\right) = 300 \times \tan(22.5^\circ) = 300 \times 0.4142 = 124.26 \text{ m}$$

2. **Curve Length ($L$):**

$$L = \frac{\pi R \Delta}{180^\circ} = \frac{\pi \times 300 \times 45^\circ}{180^\circ} = 75\pi = 235.62 \text{ m}$$

3. **Chainages of Tangent Points:**

$$\text{Chainage of } T_1(PC) = \text{Chainage of } I - T = 1250.00 - 124.26 = 1125.74 \text{ m}$$
$$\text{Chainage of } T_2(PT) = \text{Chainage of } T_1 + L = 1125.74 + 235.62 = 1361.36 \text{ m}$$

4. **Long Chord ($C$):**

$$C = 2R\sin\left(\frac{\Delta}{2}\right) = 2 \times 300 \times \sin(22.5^\circ) = 600 \times 0.3827 = 229.61 \text{ m}$$

5. **Mid-ordinate ($M$) and Apex Distance ($E$):**

$$M = R\left(1 - \cos\frac{\Delta}{2}\right) = 300 \times (1 - \cos 22.5^\circ) = 300 \times (1 - 0.9239) = 22.84 \text{ m}$$
$$E = R\left(\sec\frac{\Delta}{2} - 1\right) = 300 \times (1.0824 - 1) = 24.72 \text{ m}$$

# Summary & Key Formula Cheat Sheet I

## Vertical Curves Formula Summary

- **Algebraic Grade Change:** $N = g_1 - g_2$
- **Tangent Offset:** $y = \frac{Nx^2}{200L}$
- **Summit Curve Elevation:** $\text{RL}_{\text{curve}} = \text{RL}_{\text{tangent}} - y$
- **Sag Curve Elevation:** $\text{RL}_{\text{curve}} = \text{RL}_{\text{tangent}} + y$
- **High/Low Point Location:** $x_m = \frac{g_1 L}{N}$ (from VPC)
- **Apex Offset:** $e = \frac{NL}{800}$
- **Summit Length ($L > SSD$):** $L = \frac{NS^2}{4.4}$

## GTU Exam Tips

- Always pay careful attention to the **algebraic sign** of gradients ($+$ for upgrade, $-$ for downgrade).
- Chainage of VPT is calculated as **Chainage of VPC** $+L$, NOT Chainage of VPI $+T$!
- On Summit curves, subtract offset from tangent RL; on Sag curves, add offset to tangent RL.

## Unit Context: Advanced Surveying Equipments

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit:** Advanced Surveying Equipments
**Lecture Topic:** Introduction and basics of Digital Theodolite; Principles of Electronic Distance Measurement (E.D.M.)

- **Lecture Objectives:**
  - Understand why the vernier transit theodolite was replaced by electronic/digital theodolites in modern practice.
  - Explain how a digital theodolite reads angles using electronic encoders instead of verniers.
  - Understand the basic physical principle by which an EDM instrument measures distance using modulated electromagnetic waves.
  - Derive the phase-shift distance measurement relation used inside an EDM unit.
  - Compare accuracy, speed and error sources of electronic angle/distance measurement vs. manual methods.
- **Prerequisite Knowledge:** Vernier reading of transit theodolite (Lecture 02), tacheometry and chain/tape distance measurement.

# Why Digital (Electronic) Theodolite? I

## Limitations of the Vernier Transit Theodolite

- Reading main scale + vernier is slow, requires good eyesight, and is prone to human reading/parallax errors.
- Least count limited practically to about $20''$–$1'$; achieving finer resolution needs bulkier optical micrometers.
- No digital record – every reading must be manually booked in a field book, introducing transcription errors.

## Digital (Electronic) Theodolite

A **digital theodolite** replaces the graduated glass circle + vernier system with an **electronic angle encoder**, and displays horizontal and vertical angles directly as a numeric readout (e.g. $124^\circ 36' 20''$) on an LCD screen.

- Eliminates vernier reading errors and human estimation of fractional divisions.
- Provides higher resolution (typically $1''$ to $5''$) with much faster observation.
- Angle data can be electronically stored, or output directly to a data recorder/computer.

# How a Digital Theodolite Reads Angles I

## Electronic Circle / Angle Encoder

A glass circle is etched with extremely fine radial graduation lines (thousands of lines around 360°). As the circle rotates relative to a fixed sensor, the angle is sensed electronically instead of read visually.

- **Incremental (Grating) Encoders:**
  - A light source (LED) shines through the graduated circle onto a photodetector.
  - As the circle rotates, alternating light/dark graduation lines interrupt the beam, generating a pulse train.
  - An electronic counter counts pulses from a fixed reference (index) mark; angle = pulse count × angular value per pulse.
  - Requires the instrument to pass an index/reference mark once after switch-on ("initialization").
- **Absolute (Coded) Encoders:**
  - Circle is etched with a unique binary/Gray code pattern at every angular position.
  - Each position generates a distinct digital code, so the exact angle is known instantly at power-on, with no initialization rotation required.
- Both horizontal and vertical circles are fitted with such encoders, giving simultaneous digital horizontal angle ($H_z$) and vertical/zenith angle ($V$) readout.

# Additional Features of Digital Theodolites I

- **Automatic Compensator (Dual-Axis Tilt Sensor):** A liquid or pendulum-based electronic sensor automatically detects and corrects small residual mis-levelling of the vertical axis, removing the corresponding angular error from the displayed reading.
- **Digital Display (LCD):** Shows horizontal angle, vertical angle, and instrument status (battery, mode) simultaneously; readable in both faces (Face Left / Face Right).
- **Angle Units Selectable:** Sexagesimal degrees ($^{\circ}\,'\,''$), Gons/Grads, or Mils, selectable from the on-board menu.
- **Data Output Port:** RS-232 or Bluetooth port to transfer angle data directly to an external data recorder or, when combined with EDM, to a total station data collector.
- **Battery Powered:** Rechargeable Ni-MH or Li-ion battery replaces the fully mechanical, battery-free operation of the vernier theodolite.

## Key Distinction

A **digital/electronic theodolite measures angles only** (horizontal and vertical). When it is combined with an in-built Electronic Distance Meter (EDM) and a microprocessor, the resulting integrated instrument becomes a **Total Station** (covered in the next lecture).

## What is EDM?

**Electronic Distance Measurement (EDM)** is a technique of measuring the distance between two points using the properties of electromagnetic waves (usually modulated infrared light or laser) transmitted between an instrument and a target, instead of physical chaining or taping.

- **Basic Components:**
  - **Transmitter:** Generates a carrier wave (infrared, laser, or microwave) modulated with a known, precise frequency.
  - **Reflector (Prism) or Reflectorless Target:** Returns the wave back to the instrument.
  - **Receiver:** Detects the returned wave and measures its phase relative to the transmitted wave.
  - **Microprocessor:** Computes the distance from the measured phase difference and displays it.
- **Two Broad Principles used in EDM instruments:**
  1. **Phase Difference (Phase-Shift) Method:** Used in virtually all modern short/medium-range infrared-based EDMs (as in total stations).
  2. **Pulse (Time-of-Flight) Method:** Used in longer-range and reflectorless laser rangefinders; measures the actual round-trip travel time of a short laser pulse.

# Principle of Phase-Shift Distance Measurement I

## Concept

The instrument transmits a continuous modulated infrared wave of known wavelength $\lambda$ towards a prism reflector at the target. The wave travels out, reflects off the prism, and returns. Because the total path ($2D$) is generally *not* an exact whole number of wavelengths, the returning wave is out of phase with the outgoing wave by a measurable fraction $\Delta\lambda$.

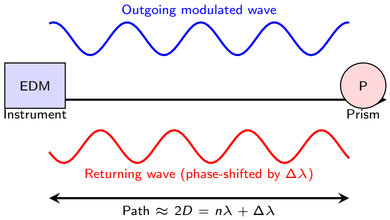



Figure: Outgoing and returning modulated waves – the measurable phase shift gives the fractional wavelength.

# Derivation of the EDM Distance Equation I

## Setting up the Relation

Let the modulated wave have wavelength $\lambda$. The double (out-and-back) path length travelled equals a whole number of wavelengths $n$ plus a fractional part $\Delta\lambda$ measured by the phase comparator:

$$2D = n\lambda + \Delta\lambda$$

where:

- $D =$ required (unknown) distance between instrument and reflector.
- $n =$ whole number of complete wavelengths in the double path (an integer, initially unknown).
- $\Delta\lambda =$ fractional wavelength corresponding to the measured phase angle $\phi$: $\Delta\lambda = \dfrac{\phi}{360^\circ}\lambda$.

## Solving for Distance

$$D = \frac{n\lambda + \Delta\lambda}{2} = \frac{\lambda}{2}\left(n + \frac{\phi}{360^\circ}\right)$$

- The phase comparator directly measures $\phi$ (hence $\Delta\lambda$) with high precision, but cannot by itself tell how many *whole* wavelengths $n$ fit in the path.
- **Resolving the ambiguity in $n$:** the instrument transmits several modulation frequencies of progressively coarser wavelength (e.g. 10 m, 1000 m pattern lengths). The coarse frequency fixes the approximate distance (and hence $n$) unambiguously; the finest frequency gives millimetre precision.

## Relation between Modulation Frequency and Wavelength

$$\lambda = \frac{c}{f}$$

where $c =$ velocity of light in the atmosphere (corrected for temperature/pressure/humidity, $\approx 3 \times 10^8$ m/s) and $f =$ modulation frequency of the carrier wave.

# Pulse (Time-of-Flight) Method & EDM Classification I

## Pulse / Time-of-Flight Principle

A short, intense pulse of laser light is transmitted; the elapsed time $t$ for the pulse to travel to the target and return is measured directly by a high-speed electronic timer:

$$D = \frac{c \cdot t}{2}$$

Used in reflectorless total stations and long-range laser rangefinders where a very short, high-energy pulse gives good return signal without needing a prism.

Table: Classification of EDM Instruments by Range

| **Type** | **Carrier Wave** | **Typical Range** |
|---|---|---|
| Short Range | Infrared light | Up to 3 km |
| Medium Range | Infrared / Laser | 3–15 km |
| Long Range | Microwave | Up to 50–150 km |

- Short and medium range infrared EDMs are the type built into modern total stations (Lecture 24 onward).
- Microwave EDM (e.g. Tellurometer) requires transceivers at both ends and is mainly of historical/geodetic interest.

## Key Takeaways

- A **digital theodolite** replaces the vernier scale with an electronic angle encoder (incremental or absolute), giving fast, error-free, digitally displayed angle readings.
- **EDM** measures distance electronically using a modulated infrared/laser wave, based on either the **phase-shift method** ($D = \frac{\lambda}{2}(n + \phi/360^\circ)$) or the **pulse (time-of-flight) method** ($D = ct/2$).
- Multiple modulation frequencies are used in phase-shift EDMs to resolve the whole-wavelength ambiguity $n$ while retaining millimetre resolution.
- Digital theodolite + EDM + microprocessor together form the basis of the modern **Total Station**, studied from the next lecture onward.

## GTU Exam Important Questions

1. Explain, with a neat sketch, the working principle of a digital (electronic) theodolite.
2. Distinguish between incremental and absolute angle encoders.
3. State the principle of EDM and derive the phase-shift distance measurement equation.
4. Why are multiple modulation frequencies used in an EDM instrument?

# Lecture 24: Agenda & Learning Objectives I

## Unit Context: Advanced Surveying Equipments

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit:** Advanced Surveying Equipments
**Lecture Topic:** Introduction and Basics of Total Station – parts, advantages/disadvantages/uses, types, Automatic Target Recognition (ATR); Surveying using Total Station – data-collection flow chart, fundamental parameters.

- **Lecture Objectives:**
  - Define a Total Station as the integration of Digital Theodolite + EDM + Microprocessor + Data Recorder.
  - Identify the major physical parts/components of a Total Station.
  - List the advantages, disadvantages and typical uses of Total Station surveying.
  - Classify types of Total Station (manual, robotic, reflectorless, prism-based).
  - Understand Automatic Target Recognition (ATR) technology used in robotic total stations.
  - Understand the fundamental parameters and data-collection flow chart used in a Total Station survey.

# What is a Total Station? I

## Definition

A **Total Station** is a single, integrated electronic/optical surveying instrument that combines an **Electronic Distance Meter (EDM)**, a **Digital (Electronic) Theodolite**, and a built-in **Microprocessor** with data storage, enabling simultaneous measurement and automatic computation of horizontal angle, vertical angle, slope distance, and 3D coordinates of any observed point.

- **Conceptually:** Total Station = Digital Theodolite (angles) + EDM (distance) + Microprocessor (computation) + Data Recorder (storage/output).
- Introduced widely from the 1970s–80s onward, it has replaced the transit theodolite + chain/tape combination as the standard field instrument for control, topographic, and construction surveys.
- All raw measurements and derived coordinates can be stored electronically and later transferred to a computer for mapping/CAD/GIS processing.

# Total Station: Integration of Sub-systems I

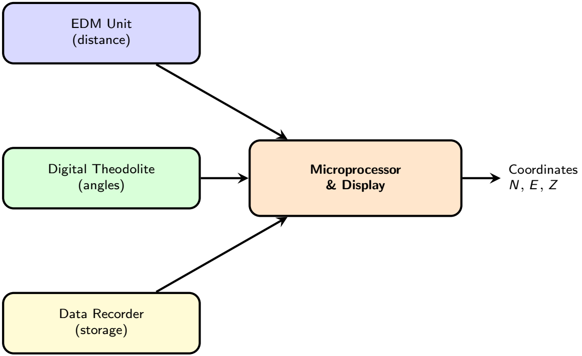



Figure: Total Station as an integration of EDM, digital theodolite, microprocessor and data recorder.

# Total Station: Integration of Sub-systems II

## Physical Parts of a Total Station

- **Telescope:** Objective and eyepiece lenses with internal focusing, carries the EDM transmitter/receiver optics coaxially with the sighting line.
- **Horizontal and Vertical Circles:** Electronic angle encoders (see Lecture 23) for $H_z$ and $V$ angle measurement.
- **Tribrach with Levelling Foot Screws:** Base unit for centering (optical/laser plummet) and levelling (circular & plate bubble, or electronic tilt sensor).
- **Keyboard & Alphanumeric Display (LCD/Touchscreen):** For entering commands, job data, and viewing measurement results, often on both faces of the instrument.
- **On-board Microprocessor:** Performs trigonometric reduction, atmospheric/PPM correction, coordinate computation, and stores programs (traverse, resection, stakeout, etc.).
- **Internal Memory / Removable Card / Data Collector Port:** Stores raw and reduced survey data; interfaces via USB, Bluetooth, or serial cable.
- **Rechargeable Battery:** Powers the EDM, encoders, display, and (in robotic models) servo motors.

## Advantages

- Simultaneous, rapid measurement of angle, distance, and elevation in one sighting – much faster than theodolite + tape.
- High accuracy: typical angular accuracy $1''$–$5''$; distance accuracy $\pm(2\text{ mm} + 2\text{ ppm} \times D)$.
- Automatic on-board computation of coordinates eliminates manual trigonometric calculation and reduces human error.
- Digital data storage removes field-book transcription errors and speeds up office data processing (direct CAD/GIS transfer).
- Built-in programs for traversing, resection, remote elevation, missing-line measurement, and stakeout increase field versatility.

## Disadvantages

- High initial cost of purchase, calibration and repair compared to a conventional theodolite.
- Requires trained operators familiar with menu-driven electronic programs.
- Battery-dependent – field work stops if power is exhausted (see Lecture 30 on battery management).
- Sensitive electronic components; vulnerable to moisture, dust, and rough handling.
- Line-of-sight instrument – still cannot measure through obstructions (unlike GNSS).

## Typical Uses

Control/traverse surveys, topographic mapping, boundary and cadastral surveys, construction layout/stakeout, road and railway alignment surveys, monitoring of structures, and volume/earthwork computation.

## Classification by Reflector Requirement

- **Prism-based (Reflector) Total Station:** Requires a corner-cube glass prism mounted on a target pole; standard mode for control and topo survey, offering long range and high precision.
- **Reflectorless Total Station:** Uses a narrow, high-intensity laser (pulse EDM) that can measure distance directly to any solid surface without a prism – essential for hazardous, inaccessible, or moving targets (building facades, cliff faces, overhead structures).

## Classification by Operation

- **Manual Total Station:** Surveyor manually points the telescope at the target and presses the measure key; one-person or two-person (surveyor + rodman) operation.
- **Motorised Total Station:** Built-in servo motors rotate the telescope automatically to a keyed-in angle, but the operator still aims/fine-points manually.
- **Robotic Total Station:** Combines servo motors with Automatic Target Recognition (ATR) and remote control, allowing a single operator to run the whole survey from the target/prism pole (see next frame).

# Automatic Target Recognition (ATR) I

## What is ATR?

**Automatic Target Recognition (ATR)** is a feature of robotic total stations in which a separate coaxial infrared laser and CCD/CMOS image sensor automatically detect, lock onto, and precisely center the crosshair on a retro-reflective prism, without the operator manually fine-sighting through the telescope.

- **Working Principle:**
  - ATR emits a narrow infrared beam coaxial with the telescope's line of sight.
  - The prism reflects this beam back; the returning image is captured by an internal CCD sensor.
  - On-board image-processing software calculates the offset between the beam centre and the prism image centre, and drives the servo motors to correct horizontal/vertical pointing until the offset is zero.
- **Lock and Track Mode:** once locked, the instrument can automatically **track** a moving prism, keeping the telescope continuously pointed at it as the rodman walks – essential for one-person robotic surveying and machine-guided construction.
- **Benefits:** Removes operator pointing error, speeds up repetitive observations, and enables single-person field operation (the operator carries a remote controller near the prism pole instead of standing at the instrument).
- **Limitation:** Requires a compatible prism target and adequate signal return; performance can degrade in strong sunlight, fog, or at very long range.

# Total Station Survey: Data-Collection Flow Chart I

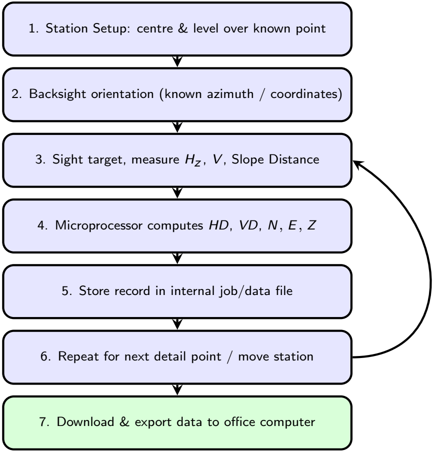



Figure: Generalised Total Station field data-collection workflow (loop repeats for every detail point).

# Fundamental Parameters in Total Station Surveying I

## Directly Observed (Raw) Parameters

- **Horizontal Angle ($H_z$):** Angle in the horizontal plane from the backsight reference direction.
- **Vertical / Zenith Angle ($V$):** Angle measured from the zenith (straight up) down to the line of sight.
- **Slope Distance ($SD$):** Straight-line distance from instrument to reflector/target measured by the EDM.

## Derived (Computed) Parameters

$$HD = SD \sin V, \qquad VD = SD \cos V$$
$$N_{\text{target}} = N_{\text{stn}} + HD \cos \theta, \qquad E_{\text{target}} = E_{\text{stn}} + HD \sin \theta$$
$$Z_{\text{target}} = Z_{\text{stn}} + HI + VD - HR$$

where $\theta$ = azimuth to target, $HI$ = instrument height above station, $HR$ = target/prism height.

## Instrument Configuration Parameters

- **Prism Constant ($PC$):** Correction for the optical offset of the prism (typically 0 or $-30$ mm).
- **Atmospheric PPM:** Correction for temperature/pressure effect on the speed of light through air.
- **Instrument Height ($HI$) and Target Height ($HR$):** Measured with a tape from the station mark to the trunnion axis, and to the prism centre respectively.

## Key Takeaways

- A Total Station integrates a digital theodolite, EDM, microprocessor and data recorder into one instrument, giving simultaneous angle, distance and coordinate measurement.
- Instruments are classified as prism-based vs. reflectorless, and manual vs. motorised vs. robotic (with ATR).
- ATR uses a coaxial infrared beam and CCD sensor to auto-lock onto a prism, enabling single-person robotic surveying.
- Field data collection follows a repeatable flow: station setup → backsight → observe → compute → store → export.

## GTU Exam Important Questions

1. Define Total Station. Draw a block diagram showing its major sub-systems.
2. List the advantages, disadvantages, and uses of a Total Station in surveying.
3. Explain the different types of Total Station based on reflector requirement and operation mode.
4. What is Automatic Target Recognition (ATR)? Explain its working principle.
5. Draw and explain the data-collection flow chart of a Total Station survey.

## Unit Context: Advanced Surveying Equipments

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit:** Advanced Surveying Equipments
**Lecture Topic:** Precautions to be taken while using Total Station; Set up of Total Station – centering, levelling, back-sighting, Azimuth marks.

- **Lecture Objectives:**
  - List handling and field precautions necessary for accurate, safe Total Station operation.
  - Perform correct instrument setup: centering and levelling over a control station.
  - Understand back-sighting and how orientation of the horizontal circle is established.
  - Understand the role and use of Azimuth marks in orienting a Total Station survey.
- **Prerequisite Knowledge:** Total Station components and fundamental parameters (Lecture 24), temporary adjustment of theodolite (Lecture 02).

# General Handling & Field Precautions I

## Instrument Handling

- Always carry the Total Station in its padded case; lift it from the tripod by the handle/carrying grip, never by the telescope.
- Never leave the instrument unattended on the tripod, especially in windy conditions or near traffic/moving machinery.
- Loosen the tripod-head clamping screw fully before removing the instrument; over-tightening the central fixing screw can damage the tribrach threads.
- Keep the instrument away from direct rain; use an umbrella or rain-cover if working in light drizzle, and never operate in a thunderstorm.

## Optical & Electronic Precautions

- Never point the telescope directly at the sun without a solar filter – this can permanently damage the EDM receiver and the operator's eye.
- Keep lens surfaces (objective, EDM window, ATR sensor) clean using a soft lens brush/cloth; never wipe with a dry cloth that may scratch coatings.
- Protect the instrument from dust and moisture; store silica-gel packets in the carrying case in humid climates.
- Do not force any rotating part (upper plate, telescope) – release the relevant clamp screw before rotating.

# Precautions During Field Survey I

- **Battery & Data:** Carry spare charged batteries; verify sufficient internal memory before starting a job (Lecture 30 covers battery management in detail).
- **Instrument/Target Heights:** Measure and record instrument height ($HI$) and prism/target height ($HR$) carefully with a steel tape at every setup – this is one of the most common sources of gross elevation error.
- **Prism Constant:** Ensure the prism constant offset set in the instrument menu matches the physical prism being used; a mismatch causes a systematic distance error on every reading.
- **Rod Verticality:** Always check that the prism pole's circular bubble is centred before taking a reading; a tilted pole introduces horizontal positioning error.
- **Environmental Correction:** Enter current temperature and pressure (PPM correction) for long, high-precision distance work.
- **Periodic Checks:** Re-observe the backsight periodically during a long survey session to confirm orientation has not drifted (due to accidental disturbance of the tripod or instrument).
- **Stability of Setup:** Avoid setting up on soft or unstable ground, near vibrating machinery, or in strong direct sunlight causing thermal distortion of the tripod legs.

## Objective of Centering

Centering means placing the vertical axis of the Total Station exactly over the ground survey station mark (peg, nail, or benchmark), just as is done for a conventional theodolite.

1. Set the tripod approximately over the station mark, legs spread firmly and evenly, tripod head roughly horizontal, at a comfortable working height.
2. Mount the Total Station on the tripod head and lightly tighten the central fixing screw (not fully, to allow small sliding adjustment later).
3. Switch on the **laser plummet** (modern instruments) or look through the **optical plummet** eyepiece (older/basic models).
4. Adjust tripod leg positions (without lifting the whole tripod) until the laser dot / optical plummet crosshair coincides exactly with the station mark.
5. Fully tighten the central fixing screw once centering is achieved.

## Note

Centering must always be checked *again* after levelling (next step), since adjusting the foot screws can slightly shift the centering. The two steps are repeated iteratively until both are simultaneously satisfied.

# Set Up of Total Station: Levelling I

## Objective of Levelling

Levelling brings the vertical (rotation) axis of the instrument truly vertical, so that horizontal angles are measured in a truly horizontal plane.

1. **Coarse Levelling:** Adjust the length of the individual tripod legs (extendable legs) until the circular ("bull's-eye") bubble is centred.
2. **Fine Levelling using Foot Screws:**
   - Rotate the upper part until the plate bubble (or electronic tilt-sensor axis) is parallel to a line joining foot screws A and B.
   - Turn A and B together, in opposite directions, until the bubble centres.
   - Rotate the instrument through $90^\circ$ so the bubble lies over the third foot screw C; turn C alone to centre the bubble.
   - Repeat and verify the bubble stays centred through a full $360^\circ$ rotation.
3. **Electronic Dual-Axis Compensator:** Most digital total stations display a graphic on-screen tilt indicator (X-tilt, Y-tilt in arc-seconds); levelling is complete when both readings fall within the compensator's working range (typically $\pm 3'$ or better) and show "Level OK".

## Iterative Centering-Levelling Cycle

Because loosening/tightening foot screws can shift the instrument slightly off the station mark, centering (laser plummet check) and levelling (bubble/compensator check) must be repeated alternately until *both* conditions are satisfied simultaneously.

# Back-sighting & Orientation using Azimuth Marks I

## Purpose of Back-sighting

Once the instrument is centred and levelled over the occupied station, its horizontal circle must be **oriented** – i.e. given a known reference direction – before any target angle or coordinate has meaning. This reference direction is established by sighting a **backsight (BS)** point of known position or known bearing.

- **Method 1 – Backsight Angle / Assumed Zero:** Sight the backsight target and set the horizontal circle reading to $0^\circ 00' 00''$ (used when only relative angles/coordinates in a local system are needed).
- **Method 2 – Backsight with Known Azimuth Mark:** Sight a reference point whose true (or grid) azimuth from the occupied station is already known – an **Azimuth Mark** – and key that known azimuth value into the instrument before pressing the orientation/set-azimuth key. The instrument then automatically offsets the horizontal circle so that every subsequent reading is a true azimuth.
- **Method 3 – Backsight by Coordinates:** Enter the known coordinates of the backsight station; the instrument computes the required azimuth internally, sights and confirms the backsight, and reports any residual coordinate misclosure as a check.

## What is an Azimuth Mark?

An **Azimuth Mark** is a permanently fixed, clearly visible and identifiable reference point (e.g. a church spire, water tank, triangulation pillar, or a purpose-built azimuth pillar) whose direction (bearing) from a control station has been accurately and independently pre-determined (often astronomically or from a higher-order control survey). Sighting it allows any instrument set up at that control station to be oriented to true north/grid north reliably and repeatably, without depending on a nearby, easily-disturbed traverse station.

# Complete Total Station Setup Sequence I

1. **Reconnaissance:** Choose a stable station location with clear intervisibility to backsight and target points.
2. **Tripod Setup:** Position tripod at working height, legs firmly planted.
3. **Centering:** Use laser/optical plummet to place instrument axis over the station mark.
4. **Levelling:** Coarse level (circular bubble/legs), then fine level (foot screws/electronic compensator).
5. **Re-check Centering:** Verify plummet is still on the mark; repeat steps 3–4 if disturbed.
6. **Instrument/Target Height:** Measure and enter *HI* (instrument height) into the job/station-setup menu.
7. **Back-sighting:** Sight the backsight/azimuth mark and orient the horizontal circle (assumed zero, known azimuth, or known coordinates).
8. **Verification:** Confirm orientation by checking computed backsight coordinates or residual angle against the known value before starting observations.

## Common Field Mistake

Skipping the re-check of centering after levelling, or failing to periodically re-observe the backsight during a long session, are the two most frequent causes of a whole day's survey data being systematically rotated or mis-positioned.

# Lecture 25 Summary & GTU Exam Focus I

## Key Takeaways

- Total Station precautions cover instrument handling, optical/electronic care, and field measurement discipline (heights, prism constant, environmental correction).
- Setup follows the same centering → levelling cycle as a theodolite, refined with laser plummets and electronic dual-axis compensators.
- Back-sighting orients the horizontal circle using an assumed zero, a known azimuth, or known coordinates.
- An Azimuth Mark is a stable, pre-determined reference direction used to reliably orient the instrument to true/grid north.

## GTU Exam Important Questions

1. List the precautions to be observed while handling and operating a Total Station in the field.
2. Explain the step-by-step procedure for centering and levelling a Total Station.
3. What is back-sighting? Explain the different methods of orienting a Total Station.
4. What is an Azimuth Mark? Why is it important in Total Station surveying?

# Lecture Overview: Total Station Operations I

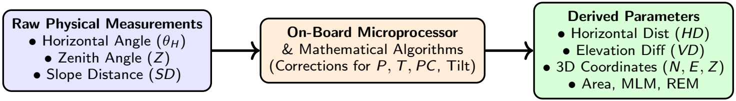
## Course & Unit Context

**Course**: Advance Surveying (DI03006021) – GTU Diploma Engineering
**Unit**: Advanced Surveying Equipments
**Topic**: Total Station Measurements, Initial Field Settings & Job File Creation

- **Key Learning Objectives**:
  - Understand raw and derived measurement capabilities of a Total Station (Angles, Slope Distance, Horizontal Distance, 3D Coordinates).
  - Study specialized embedded measurement programs: Remote Elevation Measurement (REM), Missing Line Measurement (MLM), Resection, and Stakeout.
  - Master step-by-step initial field setup: Centering over station, Coarse leveling, Fine leveling with dual-axis compensator.
  - Configure critical initial parameters: Atmospheric PPM correction, Prism constant ($PC$), Reflectorless vs Prism mode.
  - Learn memory and file management: Creating a new job, station setup ($N$, $E$, $Z$, $HI$), backsight orientation ($HR$, $Az$), and data collection.
  - Solve field numerical problems related to coordinate calculation using Total Station readings.

# Fundamental Measurement Capabilities of Total Station I

- **Definition**: A Total Station is an integrated microprocessor-controlled optical-electronic instrument combining an Electronic Distance Meter (EDM) and a Digital Electronic Theodolite.
- **Direct Raw Field Measurements**:
  - **Horizontal Angle ($\theta_H$ or $HA$)**: Measured using incremental or absolute rotary optical encoders.
  - **Vertical Angle ($\alpha$ or Zenith Angle $Z$)**: Measured relative to gravity zenith ($Z = 90^\circ - \alpha$).
  - **Slope Distance ($SD$)**: Measured using infrared phase-shift or pulsed laser transit time.
- **Automated On-Board Derived Calculations**:
  - **Horizontal Distance ($HD$)**: $HD = SD \cdot \sin Z = SD \cdot \cos \alpha$
  - **Vertical Height Difference ($VD$)**: $VD = SD \cdot \cos Z = SD \cdot \sin \alpha$
  - **Coordinates ($N, E, Z$)**: Computed instantaneously relative to occupied station.



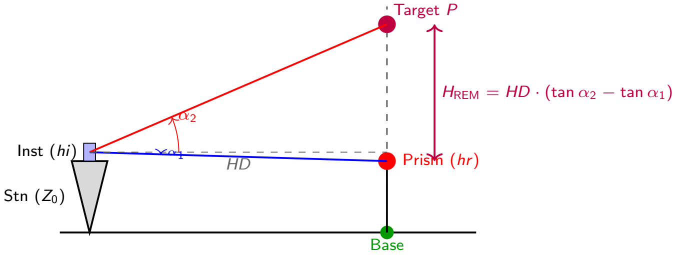
- **Coordinate Systems**: Total Station calculates 3D Cartesian coordinates $(N, E, Z)$ in a local, state, or global spatial reference frame.
- **Mathematical Formulations**: Given Occupied Station $Stn(N_0, E_0, Z_0)$, Instrument Height $hi$, Target Prism Height $hr$, Azimuth $\theta$, Zenith Angle $Z$, and Slope Distance $SD$:

## Fundamental Coordinate Equations

$$HD = SD \cdot \sin Z$$
$$VD = SD \cdot \cos Z$$
$$N_{\text{Target}} = N_0 + HD \cdot \cos\theta$$
$$E_{\text{Target}} = E_0 + HD \cdot \sin\theta$$
$$Z_{\text{Target}} = Z_0 + hi + VD - hr$$

- **Reflectorless Measurement Mode (RL)**:
  - Uses narrow laser beam (Class 2 / 3R) to measure distances to surfaces directly without a reflector prism.
  - *Applications*: Building facades, high-voltage lines, bridge piers, unstable slopes, structural monitoring.

- **Purpose**: To determine the elevation ($Z$) of an overhead or inaccessible target (e.g., power line, bridge girder, tree top) where a prism cannot be physically placed.
- **Field Procedure**:
  1. Set reflector prism directly on the ground plumb line below target point $P$.
  2. Measure target height $hr$, aim at prism base, and measure slope distance $SD_1$ and vertical angle $\alpha_1$.
  3. Unclamp vertical motion and tilt telescope up to sight inaccessible target $P$.
  4. Total station computes vertical angle $\alpha_2$ and calculates height $H_m$ automatically.



- **Purpose**: To calculate the horizontal distance, slope distance, elevation difference, and gradient between two remote target points ($A$ and $B$) without setting up the instrument over either point.
- **Modes of MLM Operation**:
  - **Radial Method ($A - B, A - C, A - D$)**: Calculates distances of points $B, C, D$ relative to a common reference base point $A$.
  - **Continuous / Chain Method ($A - B, B - C, C - D$)**: Calculates distance between consecutive pairs of points sequentially.
- **Mathematical Formulation**: If coordinates of $A(N_A, E_A, Z_A)$ and $B(N_B, E_B, Z_B)$ are determined by Total Station:

$$HD_{AB} = \sqrt{(N_B - N_A)^2 + (E_B - E_A)^2}$$

$$\Delta Z_{AB} = Z_B - Z_A, \quad SD_{AB} = \sqrt{HD_{AB}^2 + (\Delta Z_{AB})^2}$$

$$\text{Slope/Gradient} = \frac{\Delta Z_{AB}}{HD_{AB}} \times 100\%$$

# Specialized Programs: Resection & Stakeout (Setting Out) I

## Resection (Free Stationing Method)

- Used when instrument station point $S$ has unknown coordinates, but 2 or more control points ($A$, $B$, $C$) with known coordinates are visible.
- Instrument measures angles and distances to control points; on-board processor computes station coordinates ($N_S$, $E_S$, $Z_S$) and backsight orientation simultaneously using least-squares resection.
- *Advantage*: Eliminates restriction of setting up strictly on known benchmarks.

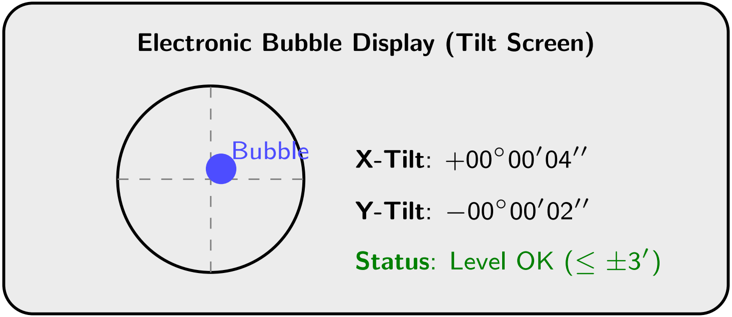
## Stakeout / Setting Out Work

- Process of transferring design coordinates ($N_d$, $E_d$, $Z_d$) from CAD drawing to actual field layout.
- Instrument compares design position with current rod position, displaying real-time field instructions:
  - Rotate angle left/right ($\Delta\theta$).
  - Move forward/backward ($\Delta HD$).
  - Cut/Fill depth ($\Delta Z$).

# Total Station Initial Field Setup: Centering & Leveling I

1. **Tripod Setup**:
   - Extend legs evenly, set tripod head horizontally over station mark at comfortable eye-level height, push leg shoes firmly into ground.
2. **Instrument Mounting & Centering**:
   - Secure Total Station on tripod head using central tribrach fixing screw.
   - Turn on **Laser Plummet** (or view through Optical Plummet) and adjust tripod legs to center laser dot directly over ground station mark.
3. **Coarse Leveling (Circular Bubble)**:
   - Adjust lengths of tripod legs individually to center the circular spirit level (Bull's Eye bubble).
4. **Fine Leveling (Plate Level / Electronic Bubble)**:
   - Rotate instrument until plate vial is parallel to line joining tribrach footscrews $A$ and $B$.
   - Turn footscrews $A$ and $B$ in opposite directions (thumb-in or thumb-out) to center bubble.
   - Rotate telescope by $90^\circ$ and adjust third footscrew $C$ to center bubble.

- **Dual-Axis Tilt Compensator**:
  - Total stations feature optical-electronic fluid tilt sensors that monitor horizontal axis tilt ($\alpha_X$) and vertical axis tilt ($\alpha_Y$).
  - Automatically corrects vertical angle index errors and horizontal angle collimation errors caused by residual level misalignment.
- **Electronic Leveling Screen**:
  - Replaces traditional tubular level vial on modern digital screens with graphic X-Y tilt values in arc-seconds ($''$).



# Environmental Settings: Atmospheric Correction (PPM) I

- **Physical Principle**:
  - EDM measures distance using light/laser velocity ($v = c/n$).
  - Refractive index ($n$) of air varies with ambient temperature ($T$) and atmospheric pressure ($P$).
  - Incorrect PPM setting causes systematic scaling errors in measured distances.
- **PPM Calculation Formula**:

$$\text{PPM} = 281.8 - \left( \frac{0.2908 \cdot P}{1 + 0.00366 \cdot T} \right)$$

  where $P$ is pressure in hPa (or mbar) and $T$ is temperature in $^\circ$C.
- **Practical Rule of Thumb**:
  - $\Delta T = 1^\circ\text{C} \implies \approx 1\,\text{ppm change.}$
  - $\Delta P = 3.6\,\text{hPa} \implies \approx 1\,\text{ppm change.}$
  - Error of 10 ppm causes 1 mm error per 100 m distance (10 mm per 1 km).

# Prism Constant & Reflector Settings I

- **Prism Constant (*PC* / Offset)**:
  - The optical origin point of a glass corner-cube prism does not coincide with its mechanical mounting center pivot bolt.
  - Distance traveled by light inside glass prism creates an effective offset (typically $-30$ mm or 0 mm).
  - **Rule**: Instrument prism constant setting must match the exact physical prism type used!
- **Reflector Types & Signal Modes**:

| Reflector Mode | Prism Offset | Range | Typical Field Use |
|---|---|---|---|
| Standard Round Prism | $-30$ mm / 0 mm | Up to 3000–5000 m | Control survey, boundary, topo |
| Mini Prism | $-17.5$ mm / 0 mm | Up to 800–1000 m | Construction setting out, detail |
| Retro Target Sheet | 0 mm | Up to 200–500 m | Structural monitoring, tunneling |
| Reflectorless (RL) | 0 mm | Up to 500–1000 m | Buildings, hazards, rock faces |

# Memory Management: Creating a New Job File I

- **Concept of Job File**: A structured database container stored in internal flash memory, SD card, or USB storage to store field measurements.
- **Job File Data Components**:
  - Header: Job Name, Date/Time, Operator Name, Scale Factor.
  - Station Data: Station ID, Occupied Coordinates ($N_0$, $E_0$, $Z_0$), Instrument Height ($HI$).
  - Orientation Data: Backsight ID, Target Coordinates/Azimuth, Target Height ($HR$).
  - Raw Data ($SD$, $HA$, $VA$) and Coordinate Data ($N$, $E$, $Z$, $Code$).
- **Step-by-Step Procedure for Job Creation**:
  1. Turn ON Total Station → Press 'MENU' key.
  2. Select 'JOB' or 'DATA MANAGEMENT' option.
  3. Select 'JOB SELECTION' → Press 'CREATE NEW JOB'.
  4. Enter alphanumeric **Job Name** (e.g., 'GTU_SURVEY_26').
  5. Configure job parameters: Units (Meters, $DDD^\circ MM' SS''$), Pressure (hPa), Temperature ($^\circ$C), Scale Factor (1.000000).

1. **Station Setup (Stn Setup)**:
   - Open active Job file → Select 'STATION SETUP'.
   - Input Occupied Station Point Name (e.g., 'STN-01').
   - Input Known Station Coordinates: Easting ($E_0$), Northing ($N_0$), Elevation ($Z_0$).
   - Measure height from station ground mark to instrument tilt axis mark using tape; enter **Instrument Height (*HI*/*hi*)**.
2. **Backsight Orientation Setup (BS Setup)**:
   - Select 'BACKSIGHT' option.
   - **Method 1: BS Angle / Azimuth**: Sight a distant reference object (e.g., benchmark or signal tower) and set horizontal circle angle to $00^\circ 00' 00''$ or known azimuth $\alpha$.
   - **Method 2: BS Coordinates**: Input known coordinates of Backsight point ($N_{BS}, E_{BS}, Z_{BS}$) and enter Target Height ($HR/hr$). Sight center of prism on BS point and press 'MEAS' / 'OBS' to verify orientation angle and coordinate check.

# Complete Total Station Field Work Workflow I

**Step 1: Physical Tripod Setup**
Centering over Ground Mark & Leveling (Circular & Electronic Bubbles)

↓

**Step 2: Environment & System Config**
Input Temp $T$, Pressure $P$ (PPM), Prism Constant ($PC$), Mode (Prism/RL)

↓

**Step 3: Job Creation**
Create New Job File, Set Units, Scale Factor, File Parameters

↓

**Step 4: Station Setup & BS Orientation**
Input $STN(N_0, E_0, Z_0)$, Measure $HI$, Sight $BS(N_{BS}, E_{BS}, Z_{BS})$ and $HR$

↓

**Step 5: Detail Data Collection / Topo**
Sight Points, Assign Codes (ROAD, TREE, POLE), Press 'MEAS' / 'REC'

↓

**Step 6: Data Transfer / Export**
Transfer CSV / DXF / ASCII files to CAD/GIS software via USB/Bluetooth

## Sample GTU Examination Problem

A Total Station is set up at station $A$ with coordinates $N_A = 1000.00\,\text{m}$, $E_A = 2000.00\,\text{m}$, Elevation $Z_A = 150.00\,\text{m}$. Measured Instrument Height $HI = 1.55\,\text{m}$. A observation is taken to a prism on target station $B$ with Target Height $HR = 1.60\,\text{m}$. The instrument records:

- Slope Distance $SD = 350.00\,\text{m}$
- Vertical Angle $\alpha = +04^\circ 30' 00''$ (Elevation above horizontal)
- Azimuth $\theta = 45^\circ 00' 00''$

Calculate: (a) Horizontal Distance $HD$, (b) Vertical Height Difference $VD$, and (c) Coordinates $(N_B, E_B, Z_B)$ of Station $B$.

## Step-by-Step Solution

1. **Horizontal Distance**: $HD = SD \cdot \cos\alpha = 350.00 \times \cos(4.5^\circ) = 350.00 \times 0.996917 = 348.92\,\text{m}$
2. **Vertical Difference**: $VD = SD \cdot \sin\alpha = 350.00 \times \sin(4.5^\circ) = 350.00 \times 0.078459 = 27.46\,\text{m}$
3. **Northing & Easting of B**:
   $N_B = N_A + HD \cdot \cos\theta = 1000.00 + 348.92 \cdot \cos(45^\circ) = 1000.00 + 246.72 = 1246.72\,\text{m}$
   $E_B = E_A + HD \cdot \sin\theta = 2000.00 + 348.92 \cdot \sin(45^\circ) = 2000.00 + 246.72 = 2246.72\,\text{m}$
4. **Elevation of B ($Z_B$)**: $Z_B = Z_A + HI + VD - HR = 150.00 + 1.55 + 27.46 - 1.60 = 177.41\,\text{m}$

# Field Precautions & Summary Checklist I

- **Common Sources of Error in Total Station Survey**:
  - *Incorrect Instrument / Prism Heights ($HI$, $HR$)*: Direct additive error in $Z$ coordinates.
  - *Mismatched Prism Constant ($PC$)*: Systematic offset in all distance measurements (30 mm error).
  - *Uncorrected Environmental PPM*: Distance scaling errors in hot/high-altitude conditions.
  - *Target Plumb Error*: Non-vertical rod holding causes horizontal positioning errors.
- **Best Practice Summary Checklist**:
  1. Always verify centering & electronic leveling status before taking readings.
  2. Always check battery voltage and optical cleanliness of lenses and prisms.
  3. Perform backsight reference checks periodically during long survey sessions.
  4. Maintain back-up of job files on external SD card / USB driver before leaving site.

# Lecture 27: Agenda & Learning Objectives I

## Unit Context: Advanced Surveying Equipments

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit:** Advanced Surveying Equipments
**Lecture Topic:** Total Station Traversing; Survey Station description, Occupied Point Entries.

- **Lecture Objectives:**
  - Understand how a Total Station automates the field and computational work of a traverse survey.
  - Compare Total Station traversing with the manual theodolite + chain traverse method learnt in Unit I.
  - Learn how to describe a survey station for unambiguous field identification and future recovery.
  - Understand what data is entered for an "Occupied Point" at each traverse station.
- **Prerequisite Knowledge:** Theodolite traversing and Bowditch adjustment (Unit I), Total Station setup and back-sighting (Lecture 25).

## How a Conventional Theodolite Traverse Works

1. At each traverse station, set up and level the theodolite; measure the included/deflection angle to the next station using the vernier scales.
2. Separately measure the horizontal distance to the next station using a chain or tape.
3. Record all angle and distance readings manually in a field book.
4. Back in the office, manually reduce bearings, compute latitudes and departures ($L = D\cos\theta$, $D_p = D\sin\theta$), sum the closing error, and apply Bowditch's Rule by hand or with a calculator/spreadsheet.
5. Plot the traverse and compute coordinates and enclosed area separately.

## Limitations

Every step above is a separate manual operation prone to reading error, arithmetic mistakes, and slow office processing – a full traverse can take hours or days to reduce by hand.

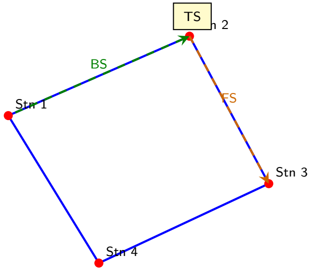
# Total Station Traversing: Concept I

## Definition

**Total Station Traversing** is the process of running a traverse (open or closed) in which, at every station, the Total Station simultaneously measures the angle *and* distance to the next station electronically, and its on-board microprocessor automatically computes and stores the coordinates of each new point – eliminating separate manual angle booking, taping, and office reduction.

- **Field Procedure at Each Traverse Station:**
  1. Set up, centre and level the Total Station over the occupied station (as in Lecture 25).
  2. Enter/confirm the occupied station's coordinates and instrument height.
  3. Back-sight the previous traverse station (or a known backsight/azimuth mark) to orient the instrument.
  4. Sight the next traverse station (foresight); the instrument measures $H_z$, $V$, and $SD$ in one observation and instantly displays/stores the computed coordinates of that point.
  5. Move the instrument to the next station and repeat, using the previous foresight point as the new backsight.
- **Automatic Closure Check:** For a closed-loop traverse, once the instrument is set up back at (or sighted to) the starting control point, the software directly reports the linear misclosure – no manual latitude/departure computation is needed.

# Sketch: Closed Traverse using Total Station I




Instrument occupies each station in turn; BS = backsight (previous stn), FS = foresight (next stn)

Figure: Closed-loop Total Station traverse – angle and distance measured together at every station.

# Comparison: Manual Theodolite Traverse vs. Total Station Traverse I

| **Aspect** | **Manual Theodolite Traverse** | **Total Station Traverse** |
|---|---|---|
| Angle Measurement | Vernier reading, manual booking | Electronic encoder, auto-displayed |
| Distance Measurement | Separate chain/tape operation | Simultaneous EDM measurement |
| Coordinate Computation | Manual latitude/departure, Bowditch by hand | Automatic, instant, on-board |
| Closure Check | Computed later in office | Available immediately in field |
| Data Recording | Field book (paper) | Digital job file (electronic) |
| Speed | Slow (hours/days to reduce) | Fast (near real-time) |
| Error Sources | Reading, booking, arithmetic errors | Instrument height/prism constant errors |

## Key Point

The underlying *geometric principle* of traversing (measure angle and distance at each station, carry forward bearing and coordinates) is unchanged – Total Station only automates the measurement and computation, removing manual error sources.

# Survey Station Description I

## Why Describe a Station?

A **station description** is a written record that uniquely identifies a survey station's location so that it can be found and reoccupied later by the same or a different survey party, and so that its coordinates can be correctly referenced in future work.

## Typical Contents of a Station Description

- **Station Name/ID:** Unique alphanumeric code (e.g. `STN-04`, `BM-12`).
- **Type of Monument:** Concrete pillar, MS nail, chiselled cross, wooden peg, benchmark plate, etc.
- **Location Narrative:** Written description referencing nearby permanent features (e.g. "3.5 m north of electric pole no. 14, on the edge of the tar road").
- **Sketch:** A simple hand sketch/tie diagram showing distances and directions to at least two or three fixed reference objects.
- **Coordinates & Elevation:** Known $(N, E, Z)$ values from the current or previous control survey, with reference datum stated.
- **Date and Surveyor:** Date established/observed and name of the surveyor, for accountability and audit trail.

# Occupied Point Entries in the Total Station Job File I

## What is an "Occupied Point"?

The **occupied point** is the survey station over which the Total Station is currently centred and levelled – i.e. the point acting as the origin for all angle and distance observations taken from that setup.

## Data Entered for the Occupied Point (Station Setup Menu)

- **Point ID / Station Name:** Selected from the job's existing point list, or newly typed in.
- **Coordinates** $(N, E, Z)$**:** Either recalled automatically (if the point already exists in the job file/control database) or entered manually for a new/unknown station.
- **Instrument Height (**$HI$**):** Vertical distance from the ground station mark to the instrument's trunnion (tilt) axis, measured by tape and entered before observations begin.
- **Point Code/Description:** Optional attribute field (e.g. `CP` for control point, `TBM` for temporary benchmark) used later for automated feature classification.

## Why Accurate Occupied Point Entry Matters

Every subsequent coordinate computed from this setup is directly built on the occupied point's entered coordinates and instrument height – any error here propagates as a constant offset into *every* point observed from that station, and (if the station is also used as backsight for the next setup) can cascade through the rest of the traverse.

## Key Takeaways

- Total Station traversing automates the classical theodolite-traverse workflow: angle and distance are measured together, and coordinates are computed and checked instantly on-board.
- The traverse's underlying geometry (bearings, latitudes/departures, closure) is unchanged – only the measurement and computation method is electronic.
- A clear, complete station description (name, monument type, sketch, ties, coordinates) is essential for reliable station recovery in future surveys.
- Every occupied-point entry (coordinates, instrument height) must be verified carefully, since errors propagate through all points observed from that setup.

## GTU Exam Important Questions

1. Explain the procedure of running a closed traverse using a Total Station.
2. Compare manual theodolite traversing with Total Station traversing.
3. What should a good survey station description contain? Why is it important?
4. What data must be entered for the occupied point at each Total Station setup, and why is accuracy critical?

# Lecture 28: Agenda & Learning Objectives I

## Unit Context: Advanced Surveying Equipments

**Course:** DI03006021 – Advance Surveying (GTU Diploma Engineering)
**Unit:** Advanced Surveying Equipments
**Lecture Topic:** Data Retrieval; Field-Generated Graphics.

- **Lecture Objectives:**
  - Understand how survey data stored inside a Total Station's job file is retrieved and transferred to a computer.
  - Identify common data transfer interfaces and standard survey/CAD file formats.
  - Understand the office data-reduction workflow from raw field data to a usable map/drawing.
  - Understand on-board (field-generated) graphical features of modern total stations and their field use.
- **Prerequisite Knowledge:** Job file creation and occupied-point/data collection (Lectures 26–27).

# Why Data Retrieval is Necessary I

## From Field Memory to Usable Output

All angle, distance and coordinate observations recorded during a Total Station survey are stored electronically inside the instrument's **job file** (internal flash memory or removable card). This raw/reduced data must be **retrieved** – downloaded out of the instrument – before it can be processed into a map, drawing, or design dataset in the office.

- Without retrieval, data remains locked inside the instrument, consuming its limited internal memory and eventually risking loss (battery failure, accidental deletion, memory overwrite).
- Retrieval is normally performed at the end of each field day (or job) as good survey practice, with the original raw file preserved unaltered as a legal/audit record.
- Retrieved data feeds directly into office software for coordinate geometry (COGO) processing, plotting, and CAD/GIS drafting.

# Data Transfer Interfaces I

## Physical / Wireless Methods of Retrieving Data

- **RS-232 Serial Cable:** The traditional wired connection between the instrument's communication port and a computer's serial/USB-serial adapter; slow but universally supported by legacy software.
- **USB Cable:** Direct USB or USB Type-C connection; instrument appears as a removable mass-storage drive, or dedicated transfer software copies job files across.
- **Removable Memory Card (SD/CF/USB drive):** Data is physically removed from the instrument and read directly on a computer with a card reader – fast and does not require the instrument itself.
- **Bluetooth / Wi-Fi:** Wireless transfer to a field controller, tablet, or office computer without physical cabling; increasingly standard on modern robotic total stations.
- **Cloud Sync:** Some modern instruments upload job data directly to a cloud storage/project-management service over a cellular or Wi-Fi connection for near real-time office access.

# Standard Survey Data & CAD Exchange Formats I

| File Type | Typical Extension | Content |
|---|---|---|
| Raw Field Data | .RAW, .SDR, .GSI | Unedited $H_z$, $V$, $SD$, station parameters – permanent record |
| Coordinate File | .CSV, .TXT, .NEZ | PointID, N, E, Z, Code in plain ASCII text |
| CAD Exchange | .DXF, .DWG | Vector drawing: points, lines, layers, symbols |
| Survey/Design Exchange | .XML (LandXML) | 3D surfaces, alignments, parcels – software-independent |

## Sample Retrieved Coordinate File (ASCII CSV)

```
201, 254120.450, 512340.120, 152.340, CP
202, 254125.800, 512348.550, 151.890, TREE
203, 254130.120, 512352.010, 151.750, EP_ROAD
```

## Good Practice

Always retrieve and archive the **raw** data file (.RAW/.SDR) first, before generating any derived coordinate or CAD file – the raw file is the only complete, editable record of exactly what was observed in the field.

# Office Data Reduction Workflow After Retrieval I

1. **Import:** Load the retrieved raw/coordinate file into survey office software (e.g. field-to-finish or CAD survey add-ins).
2. **Data Reduction:** Apply instrument/target height, prism constant, and PPM corrections if not already applied on-board; compute final reduced coordinates.
3. **Traverse/Network Adjustment:** Check angular and linear misclosure of any control traverse; distribute error (e.g. by least-squares or Bowditch's rule).
4. **Feature Coding / Linework:** Software reads point codes (e.g. `EP B`...`EP E` for "edge of pavement begin/end") to automatically draw connecting lines and insert standard symbols.
5. **Surface Modelling:** Generate a Digital Terrain Model / Triangulated Irregular Network (TIN) from the 3D points for contouring, cross-sections, and volume computation.
6. **Final Drafting:** Produce the finished CAD drawing, topographic map, or GIS layer for delivery.

# Field-Generated Graphics: Introduction I

## What are Field-Generated Graphics?

Modern total stations with a graphic touchscreen display can plot a simple, real-time **on-board graphical view** of the points measured so far during the survey, directly in the field – without waiting for office CAD software.

- **Purpose:** Lets the surveyor visually confirm, while still on site, that all required points have been captured and that the overall shape/extent of the survey looks correct – catching gross errors or missed detail before leaving the field.
- **Typical On-Screen Graphic Features:**
  - Plotted dot for every newly measured point, positioned by its computed $(N, E)$ coordinates.
  - Point ID/code label displayed alongside each plotted point.
  - Pan and zoom controls to inspect dense clusters of detail points.
  - Simple linework preview joining points sharing the same feature code (e.g. edge of road, kerb line).
  - Overlay of previously uploaded design points/lines for immediate visual comparison during a stakeout job.

# Applications & Value of On-Board Graphics I

## Field Quality Control

- Immediate visual check for missed points, gaps in a boundary/road survey, or a wildly out-of-place shot (gross blunder) caused by a wrong point ID or misread prism.
- Confirms the overall plan shape (building outline, road corridor, plot boundary) matches expectations before demobilising the survey crew.
- **Stakeout Assistance:** During construction layout, the graphic screen shows the design point and the current prism position together, helping the rodman visually converge on the exact stake location alongside the numeric $\Delta N/\Delta E$ guidance.
- **Reduces Office Rework:** Field graphics catch errors while the crew and equipment are still on site, avoiding a costly return trip that would otherwise only be discovered once the raw data is processed in the office.

## Limitation

On-board graphics are a small, low-resolution screen intended for a quick field sanity check only – they do not replace the detailed CAD/GIS processing performed on retrieved data back in the office.

# Lecture 28 Summary & GTU Exam Focus I

## Key Takeaways

- Data retrieval transfers stored Total Station job data (raw and reduced) to a computer via cable, memory card, Bluetooth/Wi-Fi, or cloud sync.
- Standard formats include raw files (`.RAW`/`.SDR`/`.GSI`), ASCII coordinate files (`.CSV`), and CAD/design exchange formats (`.DXF`, LandXML).
- Office reduction proceeds: import $\rightarrow$ correct/reduce $\rightarrow$ adjust traverse $\rightarrow$ feature-code linework $\rightarrow$ surface model $\rightarrow$ final drawing.
- Field-generated graphics give the surveyor a real-time on-board plot for immediate quality control and stakeout guidance, complementing (not replacing) office CAD processing.

## GTU Exam Important Questions

1. Explain the different methods of retrieving data from a Total Station.
2. List and describe standard survey/CAD file formats used in data transfer.
3. Describe the office data-reduction workflow from raw field data to a final drawing.
4. What are field-generated graphics? Explain their purpose and applications.

# Lecture Overview: Layout Operations & CSDS I

## Course & Unit Context

**Course**: Advance Surveying (DI03006021) – GTU Diploma Engineering
**Unit**: Advanced Surveying Equipments
**Topics Covered**: Construction Layout (Stakeout) using Total Station & Overview of Computerized Survey Data System (CSDS)

- **Key Learning Objectives**:
  - Understand the fundamental concept, objective, and significance of construction layout (setting out) in civil engineering projects.
  - Master the mathematical geometry and operational methods (Polar vs. 3D Coordinate) of Total Station stakeout.
  - Learn field procedures for setting out building grids, column centers, batter boards, road centerlines, and slope stakes.
  - Identify sources of field errors, quality control checks, and allowable tolerances in construction layout.
  - Comprehend the structure, components, and advantages of a Computerized Survey Data System (CSDS).
  - Explore data transfer interfaces, raw/coordinate file formats (.RAW, .CSV, .DXF, .LandXML), data reduction, and automated CAD/GIS mapping.

- **Definition**: Construction layout (also called *setting out* or *stakeout*) is the process of transferring design dimensions, coordinates, and elevations from engineering drawings onto the physical ground surface.
- **Surveying vs. Layout (Reverse Relationship)**:
  - *Field Surveying*: Real-World Physical Ground Features → Field Measurement → Plan / Drawing.
  - *Construction Layout*: Design Drawing / BIM Model → Field Measurement → Physical Ground Stakes.
- **Primary Objectives of Layout Work**:
  - Mark exact locations of building corners, column centers, wall footings, and structural axes.
  - Establish vertical grade lines, finished floor levels (FFL), and excavation depth markers (cut/fill).
  - Set alignment stakes for roads, railways, pipelines, bridges, and retaining walls.
- **Advantages of Using Total Station for Stakeout**:
  - Real-time vector guidance ($\Delta N, \Delta E, \Delta Z$) displayed on instrument screen.
  - High speed and millimeter-level positioning accuracy over long distances.
  - Elimination of manual taping errors and complex tape geometric calculations on site.

**1. Polar Coordinate Method**

- Station $S(N_S, E_S)$, Backsight $B(N_B, E_B)$.
- Target Design Point $P(N_P, E_P)$.
- Baseline Azimuth $\theta_{SB}$:

$$\theta_{SB} = \arctan\left(\frac{E_B - E_S}{N_B - N_S}\right)$$

- Target Azimuth $\theta_{SP}$:

$$\theta_{SP} = \arctan\left(\frac{E_P - E_S}{N_P - N_S}\right)$$

- Setting-Out Angle $\beta$:

$$\beta = \theta_{SP} - \theta_{SB}$$

- Setting-Out Distance $HD_{SP}$:

$$HD_{SP} = \sqrt{(N_P - N_S)^2 + (E_P - E_S)^2}$$

**2. Direct 3D Coordinate Method**

- Upload design coordinates $(N_P, E_P, Z_P)$ directly into Total Station job memory.
- TS measures current prism position $(N_M, E_M, Z_M)$ instantaneously.
- Real-Time Error Vectors:

**Displacement Guidance Equations**

$$\Delta N = N_P - N_M$$
$$\Delta E = E_P - E_M$$
$$\Delta Z = Z_P - Z_M$$

- Target is moved until $\Delta N = 0.000$, $\Delta E = 0.000$, and $\Delta Z = 0.000$.

# Step-by-Step Stakeout Field Procedure I

**Step 1: Station Setup**
Set TS on $S$, Level, Center
Orient to Backsight $B$

→ **Step 2: Load Stakeout**
Select SO Menu, Enter
Target Coords $(N_P, E_P, Z_P)$

→ **Step 3: Sight Angle**
Rotate TS until $\Delta H = 0^\circ 00' 00''$
Direct rodman to line

→ **Step 4: Dist & Mark**
Measure dist, adjust rod ($\Delta HD = 0$)
Drive peg, mark elevation

1. **Occupied Station & Orientation**: Set up instrument on control point $S(N_S, E_S, Z_S)$. Perform optical/laser centering and fine levelling. Sight backsight target $B$ to zero horizontal circle or set known backsight azimuth $\theta_{SB}$.
2. **Call Stakeout Program**: Open 'Stakeout' (SO) application in Total Station. Select point ID from uploaded job file or manually enter target coordinates $(N_P, E_P, Z_P)$.
3. **Angle Alignment**: Unclamp horizontal motion and turn telescope until horizontal angle difference $\Delta H = 0^\circ 00' 00''$. Clamp instrument.
4. **Prism Positioning**: Guide rodman along line of sight. Sight target prism and press MEAS/DIST.
5. **Distance Adjustment**: Read distance error $\Delta HD$ on screen:
   - Positive value ($\Delta HD > 0$): Rodman moves away from instrument ("OUT").
   - Negative value ($\Delta HD < 0$): Rodman moves towards instrument ("IN").
6. **Final Stake Marking**: When $\Delta HD = 0.000\,\text{m}$ and $\Delta Off = 0.000\,\text{m}$, mark point with wooden peg, steel pin, or masonry nail. Write Point ID and cut/fill depth on stake.

# Building Grid & Structural Axis Layout I

- **Building Layout Grid**: Major building projects use a orthogonal coordinate grid system (numerical lines 1, 2, 3... along X-axis, alphabetical lines A, B, C... along Y-axis).
- **Establishing Site Control Grid**:
  - Primary boundary control points established outside construction footprint to prevent disturbance.
  - Main grid intersection points (e.g. A-1, A-5, D-1, D-5) staked out first to form outer building envelope.
- **Batter Boards (Profile Boards)**:
  - Pegs placed directly on foundation lines get displaced during excavation.
  - Wooden batter boards built $1.5 - 2.0\,\text{m}$ outside excavation boundary line.
  - Column grid centerlines projected onto top edge of batter boards using Total Station laser pointer or optical plummet and marked with saw-cuts or nails.
- **Vertical Elevation Control**:
  - Transfer temporary benchmark (TBM) elevation to site using Total Station or precise level.
  - Mark $+1.000\,\text{m}$ datum line on concrete columns/formwork for floor level control.
  - Excavation cut/fill depth displayed as: $\Delta Z = Z_{\text{Design}} - Z_{\text{Ground}}$.

## Road Centerline & Offset Stakeout

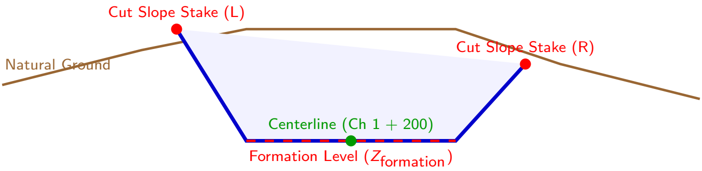
- Centerline points staked out at regular chainage intervals (10 m or 20 m).
- Offset stakes set parallel to centerline at distance $d$ (e.g., edge of carriageway, shoulder, drain).
- TS Road Software computes coordinates automatically given alignment parameters ($N_0$, $E_0$, start chainage, bearing, radius).

## Slope Stakeout (Earthwork Control)

- Slope stakes mark boundary where excavation cut or embankment fill slope intersects ground.
- Iterative stakeout process using TS height measurement until:

$$x_{\text{calc}} = w/2 + s \cdot (Z_{\text{ground}} - Z_{\text{formation}})$$

where $w$ = road width, $s$ = side slope ratio ($s$ : 1).



- **Sources of Errors in Stakeout Operations**:
  - *Instrument Setup Errors*: Miscentering over station mark, residual levelling error ($\pm 1 - 2\,\text{mm}$).
  - *Backsight Target Errors*: Imperfect sighting of backsight prism or incorrect target height ($hr$) entry.
  - *Prism Rod Non-Verticality*: Holding target prism rod out-of-plumb introduces linear positional error $\Delta e = h \cdot \sin\theta_{\text{tilt}}$ (e.g., $2^\circ$ tilt on 2 m rod creates 70 mm error!).
  - *Environmental Factors*: Atmospheric temperature/pressure changes, refraction, and optical shimmer.
- **Field Verification & Quality Assurance Checks**:
  - **Independent Diagonal Check**: Measure physical distance between adjacent newly staked pegs using tape or TS and compare against drawing dimension.
  - **Resection Check**: Station TS at an arbitrary third point and re-measure coordinates of staked pegs to confirm position.
- **Permissible Construction Tolerances (GTU / IS Code Standards)**:

| Construction Element | Horizontal Tolerance | Vertical / Elevation Tolerance |
|---|---|---|
| High-Rise Structural Columns | $\pm 3$ mm to $\pm 5$ mm | $\pm 3$ mm |
| Building Foundations / Footings | $\pm 10$ mm | $\pm 10$ mm |
| Road Carriageway & Subgrade | $\pm 15$ mm | $\pm 10$ mm |
| Major Earthwork Cut / Fill | $\pm 30$ mm to $\pm 50$ mm | $\pm 30$ mm |

# Overview of Computerized Survey Data System (CSDS) I

## Definition of CSDS

A Computerized Survey Data System (CSDS) is an integrated hardware and software environment designed to automate the entire survey data lifecycle – from electronic field data acquisition to mathematical reduction, coordinate transformation, CAD drafting, and GIS spatial analysis.

- **Evolution of Survey Data Management**:
  - *Traditional Method*: Manual field books → Manual trigonometric calculations → Manual drafting using T-squares and drawing boards. (Slow, high risk of human transcription errors).
  - *Modern CSDS Method*: Electronic Total Station / GNSS → Digital Data Collector → Automated Transfer → CAD / DTM Software → Digital Map / GIS.
- **Core Functional Components of CSDS**:
  1. **Field Hardware**: Total Station with internal memory / external field controllers.
  2. **Data Interface**: Communication hardware and protocols for transferring field observations to computers.
  3. **Office Processing Software**: Software for data reduction, error adjustment, contouring, and CAD drafting.
  4. **Spatial Output Database**: CAD drawings (.DWG/.DXF), Digital Terrain Models (DTM), and GIS layers.

# Data Transfer Interfaces & Survey File Formats I

## Physical Data Transfer Interfaces

- **RS-232C Serial Cable**: Traditional 9-pin/25-pin cable connection.
- **USB Connection**: Mini-USB / USB Type-C direct PC mount.
- **Removable Storage**: SD Cards / USB Flash Drives.
- **Wireless Transfer**: Bluetooth, Wi-Fi, Cloud cellular sync (Real-Time field-to-office).

## Standard Survey File Formats

- **Raw Data Files (.RAW, .SDR, .GSI)**: Unedited field readings (*HA*, *VA*, *SD*, stn parameters). Permanent audit trail.
- **Coordinate Files (.CSV, .TXT, .NEZ)**: ASCII text formatted as: 'PointID, Northing, Easting, Elevation, Code'.
- **CAD Exchange (.DXF, .DWG)**: Vector graphics layers, linework, and symbols.
- **LandXML (.XML)**: Standard XML schema for 3D surfaces, alignments, and parcels.

## Structure of a Standard Coordinate Data File (.CSV)

```
101, 254120.450, 512340.120, 152.340, STN_A
102, 254125.800, 512348.550, 151.890, BLDG_CORNER
103, 254130.120, 512352.010, 151.750, EP_ROAD
```

# Computerized Survey Data Processing Workflow I

- **Stage 1: Field Data Download & Import**
  - Connect field controller or insert SD card into PC.
  - Download raw field file (.RAW/.SDR) into processing software (e.g., Sokkia Link, Leica Geo Office, Trimble Business Center).
- **Stage 2: Data Reduction & Error Adjustment**
  - Compute coordinates from raw $HA$, $VA$, $SD$ readings applying instrument height, target height, and PPM corrections.
  - Perform traverse adjustment (Bowditch / Least Squares method) and check closure errors.
- **Stage 3: Automated Linework Processing (Field-to-Finish)**
  - Software interprets feature coding (e.g. 'EP B' for Edge of Pavement Begin, 'EP E' for End) to automatically draw lines, polylines, and insert block symbols (manholes, trees, utility poles).
- **Stage 4: DTM, Contouring & Earthwork Volume**
  - Generate Triangulated Irregular Network (TIN) surface from 3D ground points.
  - Interpolate smooth contour maps and generate cross-sections and profile drawings in AutoCAD Civil 3D or LISCAD.

# End-to-End Field-to-CAD Data Pipeline I

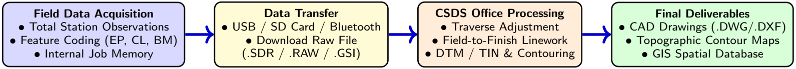



Table: Comparison: Manual Surveying vs. Computerized Survey Data System

| Parameter | Traditional Manual Surveying | Computerized Survey Data System |
|---|---|---|
| Data Recording | Handwritten field book | Electronic memory / digital data collector |
| Human Error Risk | High (transcription & calculation errors) | Minimal (automated digital data stream) |
| Processing Speed | Days to weeks | Real-time to hours |
| Outputs | Paper maps, hand-drawn plots | Digital 3D CAD models, DTM, GIS spatial layers |
| Data Archiving | Physical paper archives (perishable) | Secure digital files, cloud servers, databases |

# Data Security, QA/QC & Summary I

## Data Security & Archiving Rules

- **Raw Data Immutability**: Never alter original raw field files (.RAW/.SDR); preserve as legal project record.
- **Redundant Backups**: Store field data on primary PC, external hard drive, and cloud storage daily.
- **Metadata Standards**: Maintain project log with date, surveyor name, instrument ID, weather, and coordinate system.

## Key Takeaways of Lecture 29

- Stakeout reverses field survey to place design points on ground using $\Delta N, \Delta E, \Delta Z$ vectors.
- Batter boards and offset stakes safeguard reference control lines during site excavation.
- CSDS automates field data collection, transfer, reduction, and CAD mapping.
- Feature coding enables instant Field-to-Finish linework generation.

## GTU Exam Review Questions

1. Explain step-by-step procedure of building layout using Total Station. What are batter boards?
2. Differentiate between Polar and Coordinate methods of stakeout with suitable diagrams.
3. Describe the components and workflow of a Computerized Survey Data System (CSDS).

# Lecture Outline & Learning Objectives I

## Course Information

**Course:** DI03006021 – Advance Surveying (GTU Engineering Diploma)
**Unit:** Advanced Surveying Equipments
**Lecture 30:** Power Management, Job Planning, Estimating & Hand-Held GPS

## Learning Objectives

By the end of this lecture, students will be able to:

- Understand battery maintenance protocols and factors affecting Total Station power consumption.
- Carry out effective job planning, site preparation, and time/cost estimation for Total Station surveys.
- Explain the working principle, components, accuracy levels, and applications of Hand-Held GPS.
- Execute step-by-step field procedures for Hand-Held GPS, including waypoint marking and track logging.
- Identify error sources in GNSS/GPS navigation and implement accuracy enhancement techniques.

# 1. Total Station Battery Power Management I

- **Importance of Power Management**: Modern Total Stations rely heavily on electronic sensors, microprocessors, EDM lasers, motorized drives, and backlit displays. A depleted battery halts field operations completely.
- **Common Battery Chemistries**:
  - **Lithium-Ion (Li-ion)**: High energy density, lightweight, no memory effect, widely used in modern Total Stations (e.g., 7.4 V, 4400 mAh to 5800 mAh).
  - **Nickel-Metal Hydride (NiMH)**: Older technology, susceptible to self-discharge and moderate memory effect.
  - **Lithium Polymer (Li-Po)**: Flexible form factor, high discharge efficiency, used in compact accessories.
- **Factors Accelerating Battery Drain**:
  - **EDM Measurement Mode**: Reflectorless distance measurement (higher laser power) consumes significantly more current than standard prism mode.
  - **Display & Illumination**: High screen brightness and reticle crosshair backlighting.
  - **Motorized / Robotic Functions**: Auto-tracking, prism search, and target fine-pointing motors.
  - **Wireless Communication**: Continuous Bluetooth, Wi-Fi, or radio link with data collectors.
  - **Ambient Temperature**: Extreme cold environments ($< 0^\circ$C) reduce active chemical capacity by up to 30%–50%.

# 2. Best Practices for Battery Care & Field Maintenance I

## Charging & Storage Guidelines

- **Use Smart Chargers**: Always use manufacturer-approved smart chargers equipped with thermal shut-off and overcharge protection.
- **Storage Protocol**: Store Li-ion batteries at approximately 40%–60% charge level in a cool, dry place (15°C to 25°C) when idle for extended periods. Avoid storing fully charged or fully depleted.
- **Reconditioning**: Periodically discharge and fully recharge NiMH batteries to eliminate voltage depression (memory effect).

## Field Power Optimization

- Set instrument auto-sleep or auto-power-off timer (e.g., 5–10 minutes of inactivity).
- Lower display backlight intensity during bright daylight conditions.
- Carry minimum two fully charged spare battery packs per crew for a full day's work.
- Keep cold weather batteries in insulated pockets until insertion into the instrument.
- Consider heavy-duty external battery packs (12 V sealed lead-acid or high-capacity Li-ion power banks) for long robotic field shifts.

- **Definition**: Total Station Job Planning involves systematic pre-survey organization, site assessment, equipment configuration, and methodology selection to ensure survey accuracy, efficiency, and safety.
- **Key Phases of Job Planning**:
  1. **Desk Study & Reconnaissance**:
     - Analyze project scope, existing maps, satellite imagery, and control benchmarks.
     - Identify access routes, terrain obstacles, safety hazards, and property boundaries.
  2. **Control Network Design**:
     - Establish primary control stations with clear line-of-sight (intervisibility).
     - Select robust, stable instrument setup points free from ground movement or vibration.
     - Plan traverse loops or resection geometry for strong position determination.
  3. **Data Structure & Job File Setup**:
     - Create new job name on instrument/collector following standardized naming convention (e.g., `PROJ_2026_SECT01`).
     - Select coordinate system, projection, units (Meters/Feet, Sexagesimal degrees/Gons), and vertical datum.
     - Pre-load code libraries (e.g., `EP` for Edge of Pavement, `MH` for Manhole, `CL` for Center Line).

## Time Estimation Parameters

Total survey execution time ($T_{\text{total}}$) is calculated based on operational components:

$$T_{\text{total}} = (N_{\text{stn}} \times t_{\text{setup}}) + (N_{\text{pts}} \times t_{\text{obs}}) + t_{\text{travel}} + t_{\text{contingency}}$$

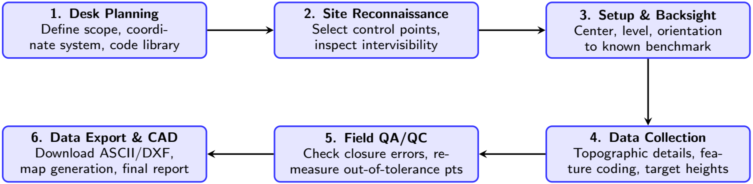
where:

- $N_{\text{stn}}$ = Number of instrument station setups.
- $t_{\text{setup}}$ = Average time per setup (typically 10–15 min for tripod setup, leveling, centering, backsighting).
- $N_{\text{pts}}$ = Total number of detail points to record.
- $t_{\text{obs}}$ = Average observation time per point (15–30 seconds for topographic detail; 2–3 minutes for control targets).
- $t_{\text{contingency}}$ = Weather/terrain buffer allowance (15%–20% of subtotal).

## Resource & Cost Estimating Components

- **Human Resources**: Party Chief / Surveyor, Instrument Operator, 1–2 Prism Rodmen, Safety/Traffic Handler.
- **Equipment Costs**: Total Station rental/depreciation rate, tripods, targets, prisms, radios, handheld GPS.
- **Operational Expenses**: Fuel, transport, lodging, field consumables (paint, wooden pegs, nails, ribbons).
- **Office Post-Processing**: Data downloading, coordinate transformation, CAD drafting, and quality check man-hours.

# 5. Total Station Job Execution Workflow I




- Systematic adherence to this workflow prevents costly re-surveys and eliminates field orientation errors.

- **What is Hand-Held GPS?**: A compact, lightweight, battery-operated satellite receiver designed primarily for real-time navigation, position locating, and low-accuracy mapping.
- **GNSS Constellations Supported**:
  - NAVSTAR GPS (USA), GLONASS (Russia), Galileo (EU), BeiDou (China), NavIC (India).
- **Operating Frequency**: Operates primarily on single-frequency L1 signal (1575.42 MHz) receiving standard coarse/acquisition (C/A) code.
- **Typical Accuracy Capabilities**:
  - **Autonomous (Standalone)**: 3 m to 10 m horizontal accuracy.
  - **SBAS Enabled (WAAS / EGNOS / GAGAN)**: 1 m to 3 m horizontal accuracy.
  - **Vertical Accuracy**: Typically 1.5 to 3 times worse than horizontal accuracy due to satellite geometry.
- **Core Functionality**: Position determination ($X$, $Y$, $Z$ or Lat, Long, Height), speed calculation, heading, waypoint storage, and track recording.

## Hardware Components

- **Antenna**: Internal patch antenna or high-sensitivity quad-helix antenna.
- **Display Screen**: Sunlight-readable color TFT/transflective screen.
- **Receiver Engine**: Multi-channel parallel processing (typically 32 to 72 channels).
- **Power Unit**: $2\times$AA batteries (NiMH/Alkaline) or internal rechargeable Li-ion pack.
- **Sensory Add-ons**: Barometric altimeter (for elevation refinement) and 3-axis electronic compass.
- **Enclosure**: Rugged, waterproof casing (IPX7 standard).

## Software Features

- **Map Rendering**: Pre-loaded base maps, topographic contour overlays.
- **Coordinate Transformation**: Real-time conversion between WGS84, UTM, and local grid systems.
- **Waypoint Manager**: Storage for 1,000–10,000 waypoints with custom icons and notes.
- **Track Log Engine**: Auto-recording of traveled paths based on time or distance intervals.
- **Area Calculation Tool**: Real-time perimeter and area computation by walking along boundaries.

# 8. Primary Applications of Hand-Held GPS in Civil Engineering I

- **1. Reconnaissance & Site Selection**:
  - Navigating to proposed site locations, locating existing benchmark pillars, identifying preliminary highway alignments.
- **2. GIS Asset Mapping & Inventory**:
  - Mapping utility assets (electric poles, manholes, hydrants, pipe valves) for municipal GIS databases.
  - Environmental mapping, tree inventory, flood line demarcation.
- **3. Field Navigation & Stakeout Assistance**:
  - Guiding survey teams through dense forests, hilly terrain, or open desert to find approximate target positions before setting up Total Station.
- **4. Preliminary Boundary Scoping**:
  - Estimating broad land parcel boundaries, reservoir submergence areas, or catchment basin limits where millimeter precision is not required.

# 9. Field Operational Procedure: Setup & Parameter Configuration I

## Step-by-Step Initial Setup Protocol

1. **Power Inspection**: Ensure fresh batteries are installed; check memory capacity and clear unnecessary old tracks.
2. **Datum Selection**: Set Geodetic Datum to **WGS 84** (World Geodetic System 1984) for standard satellite positioning, or transform to local datum (e.g., Everest 1830 for traditional Indian maps).
3. **Position Format**: Select required display format:
   - *UTM / UPS*: Universal Transverse Mercator (Easting, Northing in meters – preferred for engineering calculations).
   - *hddd.ddddd°*: Decimal Degrees.
   - *hddd° mm'ss.s"*: Degrees, Minutes, Seconds.
4. **Enable SBAS / GAGAN**: Turn ON Satellite-Based Augmentation System (WAAS/GAGAN) for real-time ionospheric correction signals.
5. **Units Setting**: Metric system (Meters, Kilometers, km/h).

# 10. Field Operational Procedure: Waypoint Marking & Track Logging I

## Procedure for Recording a Waypoint

1. **Satellite Acquisition**: Turn on device in an open area. Monitor satellite status screen until Time-To-First-Fix (TTFF) is completed and at least 4–8 satellites are locked.
2. **DOP Verification**: Check **PDOP** (Position Dilution of Precision) or **HDOP** (Horizontal Dilution of Precision). Ensure HDOP $< 2.5$ for good geometric accuracy.
3. **Positioning**: Hold the GPS level over the point of interest, avoiding body shield or overhead obstructions.
4. **Waypoint Averaging**: Enable "Average Position" mode (samples 30–100 readings over 1–2 minutes to reduce noise and multipath error).
5. **Save & Attribute**: Press `MARK / SAVE`. Assign unique Point ID, feature name, description, and save coordinates.

## Procedure for Track Logging & Area Measurement

- **Track Recording**: Select Log Method (Auto / Distance / Time interval, e.g., every 5 m or 5 seconds). Press `START TRACK`.
- **Area Calculation**: Walk carefully around the boundary of the land parcel. Upon returning to the starting point, press `STOP TRACK` → `CALCULATE AREA`. Device reports perimeter and enclosed area in $m^2$ or hectares.

- **Major Satellite & Signal Errors**:
  - **Ionospheric & Tropospheric Delay**: Atmospheric refraction slows satellite radio signals.
  - **Multipath Reflection**: Signals bouncing off tall buildings, metal structures, or rock faces before hitting antenna.
  - **Satellite Geometry (High PDOP)**: Satellites clustered closely together lead to poor intersection angles.
  - **Ephemeris & Clock Errors**: Slight drift in satellite atomic clocks or orbital path predictions.
  - **Heavy Tree Canopy**: Attenuates and scatters L1 signal, causing loss of lock or position drift.
- **Best Field Rules for Accuracy Optimization**:
  - Maintain clear, unobstructed sky view (avoid tall buildings, dense tree canopies, high-voltage power lines).
  - Use waypoint averaging for static features.
  - Keep GAGAN / SBAS mode enabled at all times.
  - Perform surveys during optimal satellite constellation availability (low PDOP window).

Table: Comprehensive Comparison Matrix

| Feature / Parameter | Total Station | Hand-Held GPS |
|---|---|---|
| **Primary Measurement** | Angle & Distance (Optical/Laser) | Satellite Radio Time Delay (Trilateration) |
| **Typical Accuracy** | High Precision (1 mm–5 mm) | Medium/Low (1 m–10 m) |
| **Line of Sight (LOS)** | Strict LOS required between station and target | Direct sky view to satellites required (no ground LOS) |
| **Setup Time** | 10–15 minutes per station (centering/leveling) | Instantaneous / 1–2 min satellite lock |
| **Operating Range** | Limited per setup (3 km–5 km to prism) | Global coverage (unlimited continuous movement) |
| **Primary Application** | Cadastral boundaries, engineering construction, structural monitoring | Reconnaissance, GIS asset mapping, navigation, preliminary route survey |
| **Cost & Complexity** | High cost, requires trained survey party | Low cost, simple single-user operation |

## Summary of Key Concepts

- Total Station battery longevity depends on EDM mode, screen backlight, and operating temperature; proper charging and maintenance ensure uninterrupted field performance.
- Systematic job planning, site reconnaissance, control network design, and time/cost estimating are vital for successful surveying projects.
- Hand-held GPS is a versatile, portable single-frequency GNSS receiver suitable for reconnaissance, asset inventory, navigation, and approximate mapping.
- Accurate field procedure requires verifying low PDOP, configuring proper datum/coordinate systems (WGS84/UTM), enabling GAGAN/SBAS, and performing waypoint averaging.

# 13. Summary & Self-Assessment Questions II

## GTU Review Questions

1. Enlist the factors affecting Total Station battery consumption and state best practices for battery maintenance. (4 Marks)
2. Explain the step-by-step job planning and time estimation procedure for a Total Station topographic survey. (7 Marks)
3. Describe the working principle and main civil engineering applications of a Hand-Held GPS. (4 Marks)
4. Write down the field operational procedure for marking a waypoint and calculating area using a Hand-Held GPS. (7 Marks)
5. Compare Total Station and Hand-Held GPS with respect to accuracy, line of sight, setup time, and application areas. (4 Marks)

## This is the Final Lecture of the Course

Having covered Hand-Held GPS, this lecture closes the syllabus of **DI03006021 – Advance Surveying**. Below is a recap of the four units studied across the semester.

1. **Unit I – Theodolite Traverse:**
   - Transit theodolite construction, reading of main/vernier scales, least count, temporary adjustments (centering, levelling, elimination of parallax).
   - Methods of traversing (included angle, deflection angle, direct/reverse), and traverse computation, adjustment (Bowditch rule), and area from coordinates.
2. **Unit II – Trigonometric and Tacheometric Survey:**
   - Trigonometric levelling for heights and distances of inaccessible objects.
   - Tacheometry: stadia principle, distance formula $D = Ks + C$, anallatic lens, fixed-hair and movable-hair methods.
3. **Unit III – Curves:**
   - Simple circular curves: designation, elements, setting-out (tangent length, chainages of PC/PT).
   - Transition and compound curves; vertical curves (summit and valley/sag) for smooth gradient change.
4. **Unit IV – Advanced Surveying Equipments:**
   - Digital theodolite and EDM principles; Total Station components, types, ATR, setup, measurement, traversing, data handling, and construction layout.
   - Hand-held GPS for reconnaissance-grade navigation and asset mapping.

## Overall Learning Outcome

By course end, a student should be able to select the appropriate surveying instrument and method – from a simple theodolite traverse to a full Total Station / GNSS-based workflow – for any given control, topographic, or construction-layout survey task, and process the resulting data into usable maps, coordinates, and setting-out information.