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Indian Institute of Technology Kanpur

Department of Civil Engineering

Survey Camp Report

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Course Code: **CE673A**

Course Name: **Instrumentation, laboratory and Field Practices in Geoinformatics**

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Subham Raj

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CHAPTER 1

Introduction

Surveying is the process of determining the relative position of natural and man- made features on or under the earth's surface, the presentation of this information either graphically in the form of plans or numerically in the form of tables, and the setting out of measurements on the earth's surface. It usually involves measurement, calculations, the production of plans, and the determination of specific locations. The results of surveys are used to map the earth, prepare navigational charts, establish property boundaries, develop data of land used and natural resource information etc. Further survey maintains highways, railroads, buildings, bridges, tunnels, canals, dams and many more. There are two major categories of surveying:

1. Plane surveying deals with areas of limited extent and it is assumed that the earth's surface is a plane and therefore no corrections necessary for the earth's curvature.

2. Geodetic surveying is concerned with determining the size and shape of the earth and it also provides a high-accuracy framework for the control of lower- order surveys. The highest standards of accuracy are necessary. Geodetic surveys cover relatively large areas (eg a state or country) for which the effects of earth curvature must be considered.

Surveying is based on several principles, including:

- **Consistency**– the principle involving consistency means being consistent in the instruments, readings, methods used, and the notations made from observations to gain a level of accuracy

- **Accuracy**– in surveying, to be accurate you have to use the proper instruments and methods based on the level of accuracy required Location of a point through measurement using two points of reference– this principle means the desired point to be surveyed has to be located by taking measurements from two or three points of reference

- **Independent check**– This principle of independent checks is to prevent errors in your work. Each measurement taken in the field must be re-checked so there are no mistaken observations that get passed without notice
- **Working from whole to part**– the survey has to be carried out from the whole to part as a system of control so it covers the entire area with a high degree of precision. This principle will prevent undue errors and localize and control minor errors

CHAPTER 2

Objective

The aim of conducting this exercise is to create a topographic map of Arogydham campus located in Chitrakoot, a pilgrimage centre and a nagar panchayat in the Satna district in the state of Madhya Pradesh, India. The exercise also demands production of a GIS based navigational map covering the important features of the Chitrakoot city.

2.1 Work Description

Table 1: Timeline

Date	Schedule
27/11/2022	1. Briefing by Prof. Onkar Dikshit 2. Reconnaissance 3. Establishing control points
28/11/2022	JUNO Data Collection
29/11/2022	Total Station Traversing
30/11/2022	Levelling by Auto Level
1/12/ 2022	GNSS observations at all control points
2/12/2022	Feature Mapping with Total Station
3/12/2022	Feature Mapping with Total Station
4/12/2022	Site Seeing
5/12/2022	Road Profiling with Total Station
6/12/2022	Departure to llTK

2.2 Location Description

- **Latitude** = 25.1562° N
- **Longitude** = 80.8631° E
- **Altitude** = 140 m (Highest: 700 m)
- **UTM Zone**: 44N
- **Nearest Railway Station**: Chitrakot Railway Station

Figure 1: Route Map to AarogyaDham, Chitrakoot

Figure 2: AarogyaDham Campus

CHAPTER 3

Reconnaissance

Reconnaissance in surveying and mapping refers to the preliminary survey or examination of a given area before the actual survey work is carried out. It is an essential step in the surveying process, as it provides the surveyor with critical information about the area they are mapping.

During reconnaissance, a general walk through was done of the area to be surveyed, making note of any relevant features such as roads, buildings, natural landmarks, and boundaries gathering information about the area's topography and other physical characteristics that could affect the surveying process. This information is then used to plan the actual survey work and to establish control stations so that intervisibility can be maintained and maximum features can be collected while mapping.

Consumable Items: Paper, Paint, Pegs and Hammer for marking the control stations

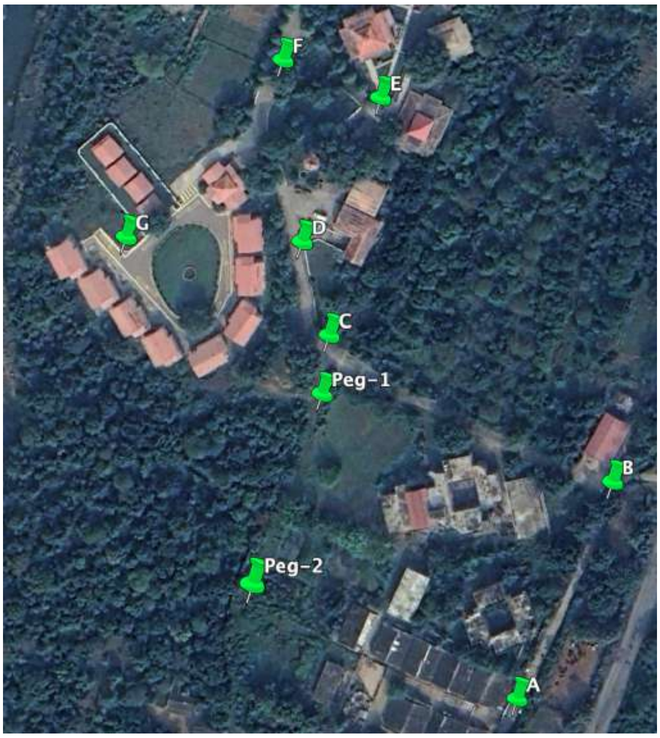


Figure 3: Proposed Control Stations

4

CHAPTER

Juno Map

4.1 Objective

The purpose of conducting this exercise is to produce a GIS based navigational map of Chitrakoot city using Juno 3B device for proposed path shown below:

Figure 4: Route Map

4.2 Instrument and Software Description

- **Juno 3B** GNSS is a handheld Global Navigation Satellite System (GNSS) receiver manufactured by Trimble. It is designed for field applications that require location data, such as surveying, mapping and producing small scale maps. The Juno 3B GNSS receiver is capable of both pseudoranges and carrier phase measurements, and can be configured to use either or both depending on the specific application and requirements.

Figure 5: Juno 3B Device

- **ArcGIS Pro** is a full-featured professional desktop GIS application from Esri. With ArcGIS Pro, we can explore, visualize, and analyze data; create 2D maps and 3D scenes.

4.3 Procedure

4.3.1 Working with Juno 3B:

1. Battery was connected to the device.
2. In menu option "TerraSync" was opened and standard edition was selected.
3. In setup option, following selections were made:
    - Coordinate System: UTM
    - Zone: 44N
    - Datum: WGS 1984
4. Logging Interval was set to 1 seconds.
5. In setup option, Data was selected and a new file was created.
6. A new window named "Data" appears with following three type of features:
    - Point generic (for point features)
    - Line generic (for line features)
    - Area generic (for area features)

7. For every feature collected, a comment was added. While taking area or line features, point features was also taken by selecting Options ⟶ Nest ⟶ Point generic.

8. All three types of data was collected on the route map provided starting and ending at Aarogyadham Campus.

4.3.2 Working with ArcGIS Pro:

1. Data obtained in the form of shape files (line, area and point features) from the device was transferred and added to ArcGIS Pro.
   Map ⟶ Add Data ⟶ Select all shape files.

2. All the point data resembling same type of feature was categorised into one class and similarly several classes were created using attribute query.
   Open Attribute data ⟶ Select by attributes ⟶ Where ⟶ Categorising to corresponding class name

3. Selecting the class property from drawing order, choosing the symbology option and selecting the appropriate symbol for that class.

4. Similarly all classes were assigned a proper symbology.

5. All cartographic elements such as coordinate system, legend, scale, north arrow, title, etc are created and exported to pdf format in 600 dpi.

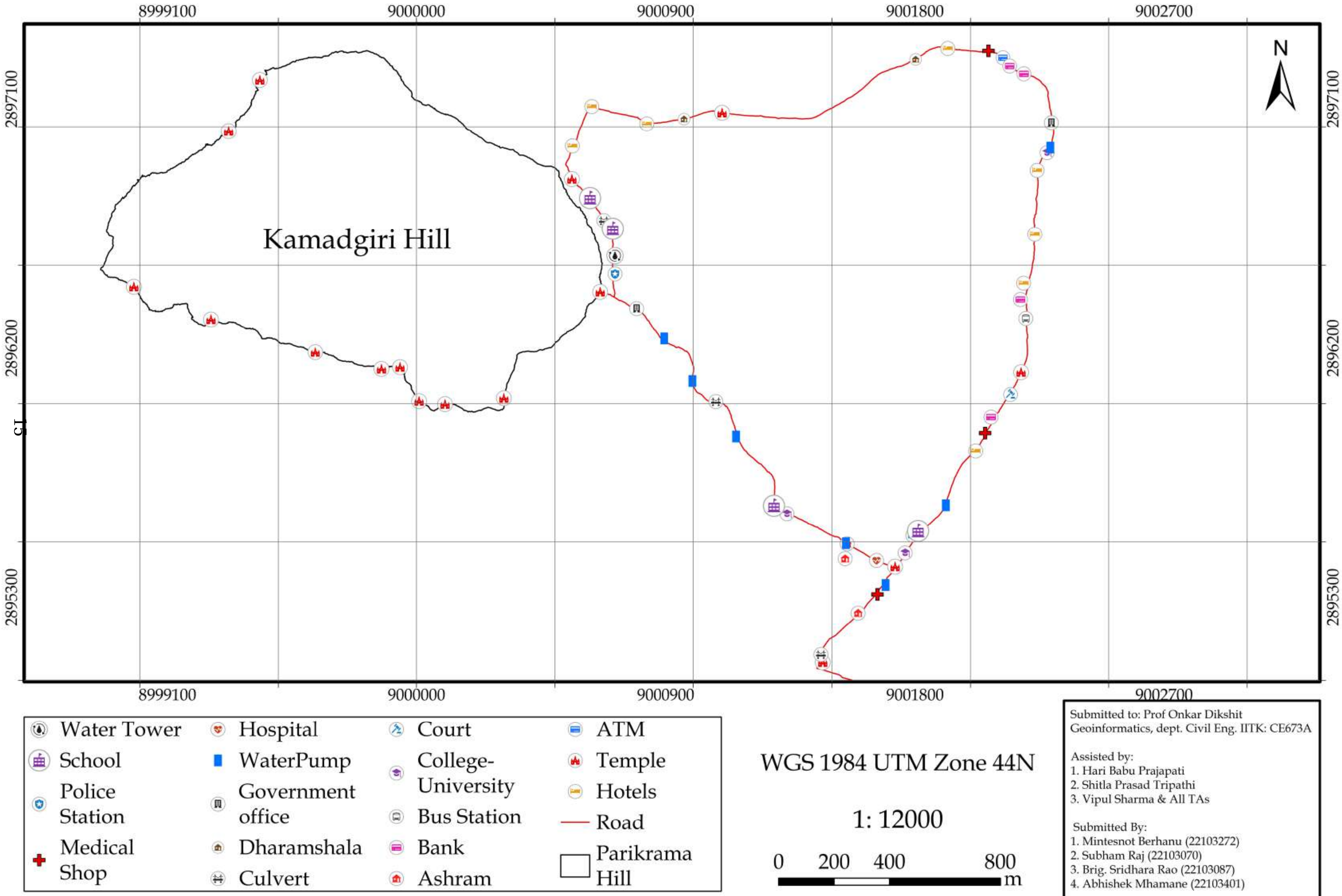
4.4 Conclusion

Accuracy of the observations is obtained as part of data processing report and it shows the estimated accuracies for all the corrected positions. 17764 (99.42 %) of 17868 selected positions were code corrected by post-processing with the following accuracy.

Range	Percentage
0-5cm	-
5-15cm	-
15-30cm	-
30-50cm	-
0.5-1m	-
1-2m	0.13
2-5m	23.09
>5m	76.78

Table 2: Differential Correction Summary

Juno GPS Map



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CHAPTER 5

Levelling

5.1 Introduction

Levelling is a branch of surveying, the object of which is: i) to find the elevations of given points with respect to a given or assumed datum, and ii) to establish points at a given or assumed datum. The first operation is required to enable the works to be designed while the second operation is required in the setting out of all kinds of engineering works. Levelling deals with measurements in a vertical plane.[2]

5.2 Objective

Using GNSS we can only get ellipsoidal heights, but for generating geoid models or a contour map, we need orthometric height. The aim of this exercise is to carry out levelling for all the control stations and establishing the elevations which can be further interpolated to have a contour map.

5.3 Terminologies

- **Level surface**: A level surface is defined as a curved surface which at each point is perpendicular to the direction of gravity at the point. The surface of a still water is a truly level surface. Any surface parallel to the mean spheroidal surface of the earth is, therefore, a level surface.[2]

- **Level line:** A level line is a line lying in a level surface. It is, therefore, normal to the plumb line at all points.[2]

- **Horizontal plane:** Horizontal plane through a point is a plane tangential to the level surface at that point. It is, therefore, perpendicular to the plumb line through the point.[2]

- **Horizontal line:** It is a straight line tangential to the level line at a point. It is also perpendicular to the plumb line.[2]

- **Vertical line:** It is a line normal to the level line at a point. It is commonly considered to be the line defined by a plumb line.[2]

- **Datum:** Datum is any surface to which elevation are referred. The mean sea level affords a convenient datum world over, and elevations are commonly given as so much above or below sea level. It is often more convenient, however, to assume some other datum, specially, if only the relative elevation of points are required.[2]

- **Elevation:** The elevation of a point on or near the surface of the earth is its vertical distance above or below an arbitrarily assumed level surface or datum. The difference in elevation between two points is the vertical distance between the two level surface in which the two points lie.[2]

- **Vertical angle:** Vertical angle is an angle between two intersecting lines in a vertical plane. Generally, one of these lines is horizontal.[2]

- **Mean sea level:** It is the average height of the sea for all stages of the tides. At any particular place it is derived by averaging the hourly tide heights over a long period of 19.6 years (nutation period).[2]

- **Bench Mark:** It is a relatively permanent point of reference whose elevation with respect to some assumed datum is known. It is used either as a starting point for levelling or as a point upon which to close as a check.[2]

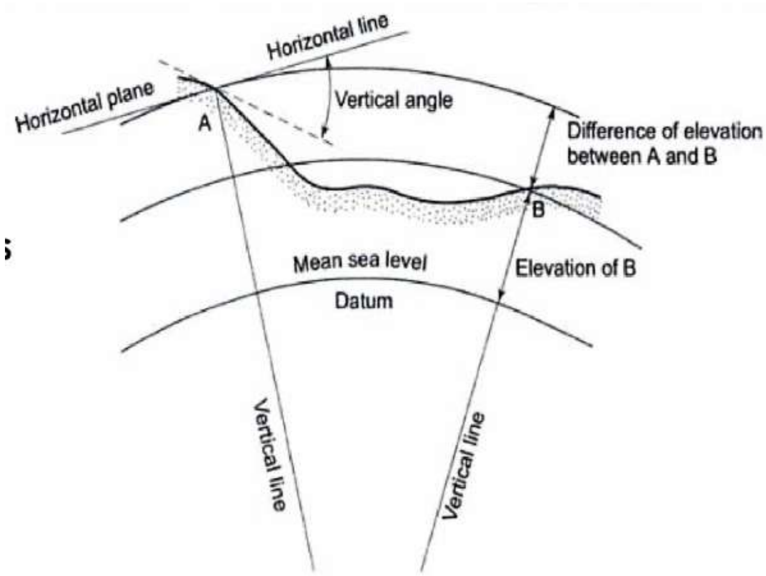


Figure 6: Levelling Components

5.4 Principle

Height of the instrument (HI): It is the elevation or reduced level of the line of sight with respect to the datum.

$$\mathrm{HI} = \mathrm{BM} + \mathrm{BS}$$

Backsight (BS): It is a staff reading taken on a point of known elevation, as on a benchmark or a change point. This is also called as a plus sight. The backsight is the first staff reading taken after the level is set up and leveled at the point.
Foresight (FS): It is a staff reading taken on a point whose elevation has to be determined through levelling process. It is also known as minus sight. The foresight is also taken towards a change point. It is the last reading taken before the instrument is shifted. All prior readings are termed as **Intermediate Sight (IS)**.

$$\text{Change in Elevation, } \Delta\mathrm{H} = \mathrm{FS} - \mathrm{BS}$$

Change point or Turning point (TP): It is a point denoting the shifting of the level. It is a station at which both BS and FS readings are taken.

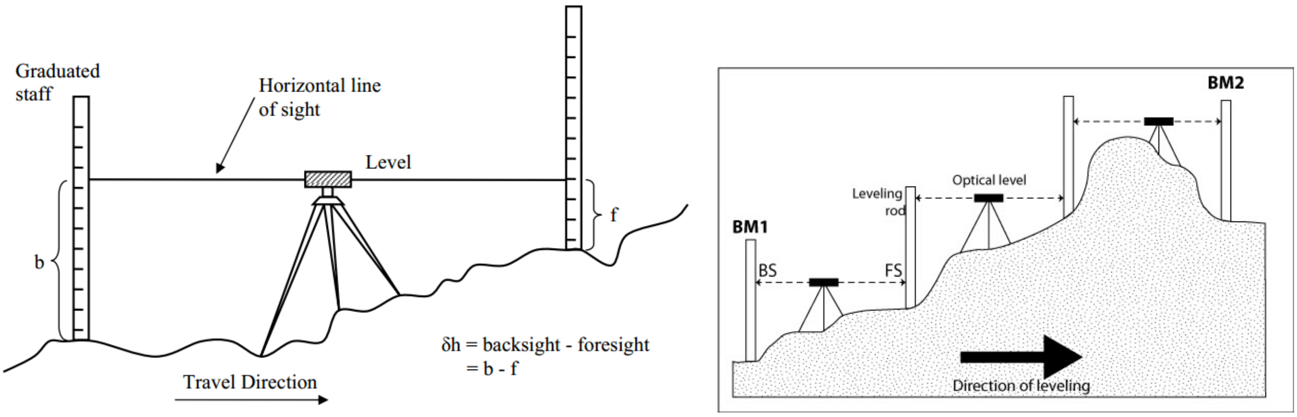


Figure 7: Levelling Principle

5.5 Instrument Description

5.5.1 Auto-Level

An automatic level is an optical instrument used to establish or verify points in the same horizontal plane in a process known as leveling and is used in conjunction with a leveling staff to establish the relative heights levels of objects or marks.

5.5.2 Levelling Staff

The levelling staff is a box portion of aluminium or wood that can be telescoping, hinging, or add sections to stretch to 3 or 5 metres in height. A graduated scale is connected to one face for reading with the level telescope's cross-hairs.

5.5.3 Tripod

Auto-level is mounted on the tripod which is portable and provide support and stability along both the side-to-side and up-and-down axis of motion with three adjustable legs.

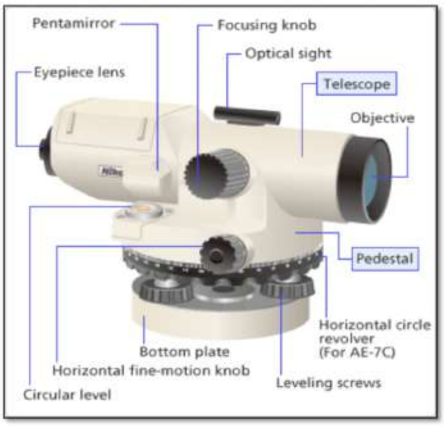


Figure 8: Auto-Level

Figure 9: Level Staff

5.6 Procedure

Temporary Adjustment: Setting ⟶ Levelling ⟶ Focussing

1. Tripod stand was set firmly on the ground so that its top is at a convenient height and instrument was mounted on it.
2. Leveling of the instrument was done to make the vertical axis of the instrument truly vertical.
3. Clamp was loosened using the levelling screws and telescope was turned until the bubble axis was parallel to the line joining any two screws.
4. Two screws were turned inward or outward equally and simultaneously till bubble was centred.
5. Telescope was turned by 90° so that it lies over the third screw and level the instrument by operating the third screw.
6. Telescope was turned back to its original position and bubble was checked. Steps 2 to Step 4 was repeated till the bubble was centred for both positions of the telescope.
7. Level Staff was kept on a known Benchmark (BM) and using the eye-piece, the cross-wires was focused to the staff and corresponding reading was taken as BS.
8. Readings corresponding to upper stadia and lower stadia was also taken in order to measure the distance from instrument.

$$D = KS + C$$

where K is multiplying constant, S is stadia difference, C is additive constant and D is the distance between instrument and staff. (For analletic lens K = 100 and C = 0).

$$\text{Distance, } D = 100 \times (\text{Upper Stadia Reading} - \text{Lower Stadia Reading})$$

9. Staff was then taken to a intermediate point (transiting in the direction of next control station) and instrument was made to focused on staff and corresponding reading was taken as FS.

10. All the steps were repeated and readings were taken at all the control stations in the proposed traverse loop.

5.7 Observation Chart

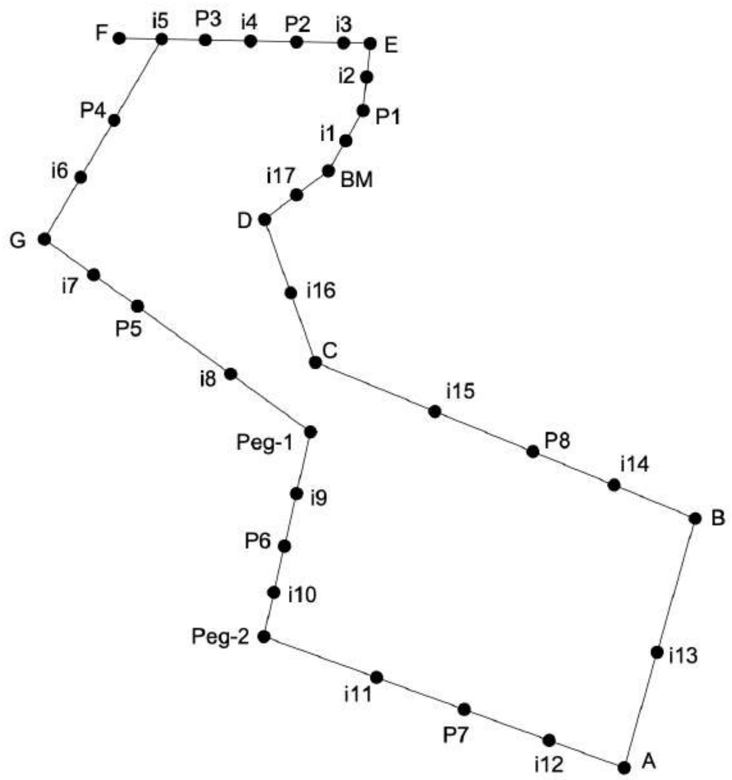


Figure 10: Level Net (i: Instrument setups, P: Intermediate points)

Table 3: Levelling Field Book

Station	B.S(m)	I.S(m)	F.S(m)	Upper Stadia (m)	Lower Stadia (m)	HD (m)	HI (m)	RL (m)
BM	0.315			0.399	0.239	16	100.315	100
P1	1.507		1.35	1.433	1.267	16.6	100.315	98.965
				1.597	1.417	18	100.472	
PG1-E	0.206		1.416	1.481	1.349	13.2	100.472	99.056
				0.276	0.135	14.1	99.262	
P2	1.164		3.134	3.202	3.065	13.7	99.262	96.128

				1.218	1.111	10.7	97.292	
P3	2.26		1.442	1.534	1.35	18.4	97.292	95.85
				2.344	2.175	16.9	98.11	
PG1-F		1.637		1.692	1.581	11.1	98.11	96.473
P4	1.101		2.18	2.291	2.068	22.3	98.11	95.93
				1.279	0.923	35.6	97.031	
PG1-G	2.154		1.921	2.005	1.837	16.8	97.031	95.11
				2.314	1.993	32.1	97.264	
P5	2.478		1.389	1.533	1.248	28.5	97.264	95.875
				2.608	2.348	26	98.353	
PEG-1	0.736		1.502	1.599	1.406	19.3	98.353	96.851
				0.788	0.685	10.3	97.587	
P6	1.555		2.174	2.218	2.133	8.5	97.587	95.413
				1.674	1.439	23.5	96.968	
PEG-2	3.509		0.965	1.088	0.842	24.6	96.968	96.003
				3.806	3.211	59.5	99.512	
P7	2.221		0.389	0.579	0.202	37.7	99.512	99.123
				2.29	2.151	13.9	101.344	
PG1-A	1.033		0.6	0.661	0.539	12.2	101.344	100.744
				1.22	0.848	37.2	101.777	
PG1-B	1.058		1.56	1.789	1.331	45.8	101.777	100.217
				1.22	0.94	28	101.275	
P8	1.181		2.051	2.191	1.911	28	101.275	99.224
				1.349	1.011	33.8	100.405	
PG1-C	2.279		1.57	1.782	1.359	42.3	100.405	98.835
				2.398	2.163	23.5	101.114	

PG1-D	1.601		1.503	1.629	1.379	25	101.114	99.611
				1.647	1.555	9.2	101.212	
BM			1.195	1.237	1.154	8.3	101.212	100.017

5.8 Checks

5.8.1 Height of Instrument Method:

The difference between the sum of the back sights and the sum of the fore sights should be equal to the difference of the first and the last R.Ls. This check verifies the calculation of R.Ls. of the planes of collimation and of the change points only. There is no check on the reduction of R.Ls. of the intermediate points.

$$\Sigma(\mathrm{BS}) - \Sigma(\mathrm{FS}) = \mathrm{Last\ RL} - \mathrm{First\ RL} = 0.017\ \mathrm{m}$$

5.8.2 Rise and Fall Method:

In this method, the difference between consecutive points is calculated by comparing each point after the first with that immediately preceding it. The difference of their staff reading indicates rise or fall according as any staff reading is smaller or greater than that at the preceding point. The R.L. of each point is then found by adding rise or subtracting fall to or from the R.L. of the preceding point.

$$\Sigma(\mathrm{BS}) - \Sigma(\mathrm{FS}) = \Sigma\mathrm{Rise} - \Sigma\mathrm{Fall} = \mathrm{Last\ RL} - \mathrm{First\ RL} = 0.017\ \mathrm{m}$$

5.9 Sources of Error

- The main source of instrumental error is residual collimation error.Keeping the horizontal lengths of the backsights and foresights at each instrument position equal will cancel this error. Where the observational distances are unequal, the error will be proportional to the difference in distances.[3]
- Staff not held vertical, levelling involves vertical measurements relative to a horizontal planes so it is important to ensure that the staff is held strictly vertical.[3]
- Natural errors such as refraction and curvature leading to bending of horizontal line.

5.10 Quality of work

Permissible error for levelling is given by:

$$\mathrm{E} = \mathrm{c}\sqrt{\mathrm{n}}$$

where c = 5mm and n is the number of setups of auto-level instrument.
Using the above formula, the permissible error was calculated as 20.61 mm. The above levelling exercise was completed with 17 setups with 17mm error in closing the Benchmark which is well within the permissible limit and results can be accepted.

5.11 Adjustment of Level Net using Least Squares Method

Table 4: Raw Observations

From	To	Height Difference (m)	Lengths (m)
BM	P1	$H_1 = 1.035$	32.6
P1	E	$H_2 = -0.091$	31.2
E	F	$H_3 = 2.583$	47.1769
F	G	$H_4 = 1.363$	85.0791
G	Peg-1	$H_5 = -1.741$	104.8657
Peg-1	Peg-2	$H_6 = 0.848$	66.9431
Peg-2	A	$H_7 = -4.741$	122.3941
A	B	$H_8 = 0.527$	82.6604
B	C	$H_9 = 1.382$	131.0812
C	D	$H_{10} = -0.776$	48.4412
D	BM	$H_{11} = -0.406$	17.5
		Misclosure, W = -0.017	

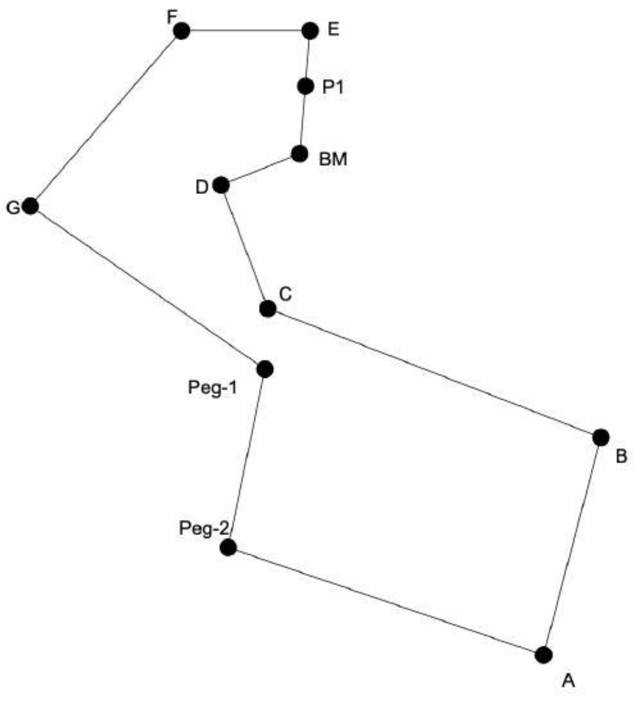


Figure 11: Level Net

Sum of all height differences originating from BM (RL=100 m) should be equal to zero. Therefore the mathematical model for condition equation becomes:

$$F(X_a, L_a) = 0$$

$$\text{and } \hat{H}_1 + \hat{H}_2 + \hat{H}_3 + \hat{H}_4 + \hat{H}_5 + \hat{H}_6 + \hat{H}_7 + \hat{H}_8 + \hat{H}_9 + \hat{H}_{10} + \hat{H}_{11} = 0$$

From condition equation method of form BV+W = 0, we can compute:

- **Step: 1** P = 1/Length
- **Step: 2** B = [1 1 1 1 1 1 1 1 1 1 1]
- **Step: 3** $M = BP^{-1}B^{T}$
- **Step: 4** $K = -M^{-1}W$
- **Step: 5** Residual, $V = P^{-1}B^{T}K$
- **Step: 6** Adjusted Observations = Height Difference + V

Table 5: Results

From	To	Height Difference (m)	Residuals	Adjusted Observations	Adjusted Elevations
BM	P1	1.035	0.00071979	1.03571979	BM = 100
P1	E	-0.091	0.00068888	-0.09031112	P1 = 98.96428021
E	F	2.583	0.00104165	2.58404165	E = 99.05459133
F	G	1.363	0.00187851	1.36487851	F = 96.47054968
G	Peg-1	-1.741	0.00231539	-1.73868461	G = 95.10567117
Peg-1	Peg-2	0.848	0.00147808	0.84947808	Peg-1 = 96.84435578
Peg-2	A	-4.741	0.00270241	-4.73829759	Peg-2 = 95.9948777
A	B	0.527	0.00182511	0.52882511	A = 100.73317529
B	C	1.382	0.00289422	1.38489422	B = 100.20435018
C	D	-0.776	0.00106956	-0.77493044	C = 98.81945596
D	BM	-0.406	0.00038639	-0.40561361	D = 99.5943864
Misclosure		**= -0.017**		**=-1E-08**	

5.12 Conclusion

Quality of the work lies within the permissible limit and it can be shown that error propagated from various sources can be well adjusted using least squares approach. The results obtained can be used for geoid modelling for the area using several interpolation techniques such as IDW interpolation, Krigging, etc. thus defining the vertical datum of the region. Digital Elevation Model (DEM) can also be produced which can be used in future for modelling water flow, landslide detection, etc

CHAPTER 6

Traversing

6.1 Introduction

A traverse is a series of connected lines whose lengths and directions are to be measured and the process of surveying to find such measurements is known as traversing. In general, chains are used to measure length and compass or theodolite are used to measure the direction of traverse lines but for this exercise electronic transit theodolite known as "Total Station" was used.

6.2 Objective

The aim for this exercise is to find out the horizontal angles between the sides of proposed polygon and distances between the control stations.

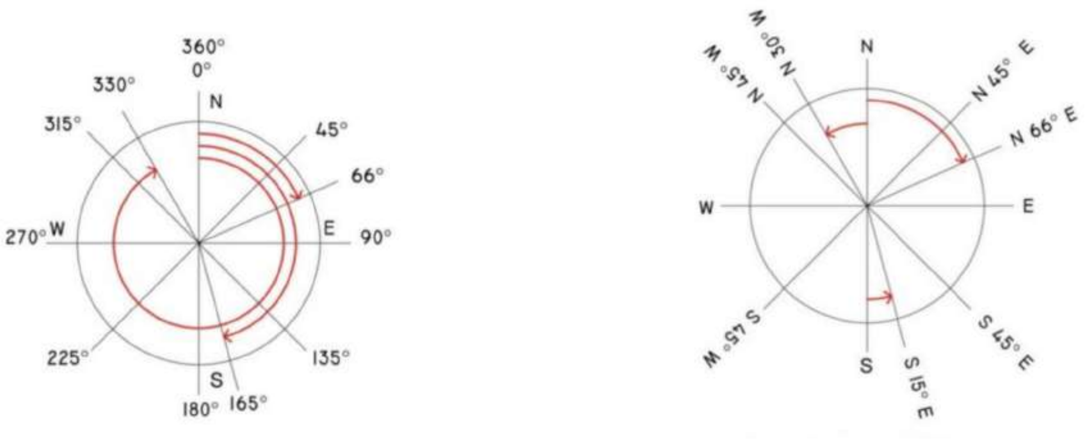
6.3 Theory

Traversing consists of a series of connected lines the bearing of the lines as well as horizontal distance is measured by a total station. This survey does not require any formulation of a triangle network and this compass survey is suitable for large areas, full of many details and undulating.

6.3.1 Types of Bearing System

- **Whole Circle Bearing System** In this system, the bearing of the line is measured from the north in clockwise direction.

- **Quadrantal Bearing System** In this system, the bearing of the line is the acute angle which the lines makes with the meridian in both clockwise and anticlockwise direction.



(a) Whole Circle Bearing System

(b) Quadrantal Bearing System

Figure 12: Types of Bearing Systems

6.3.2 Fore Bearing and Back Bearing

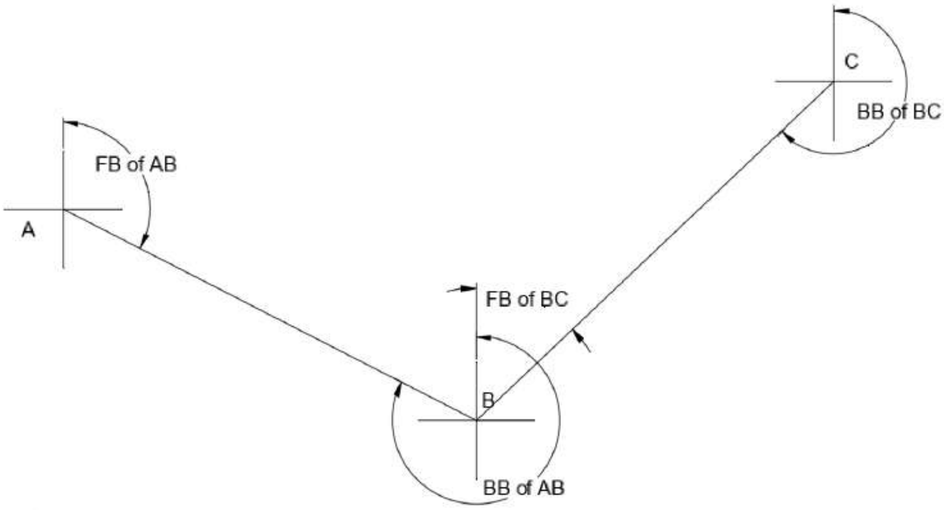


Figure 13: Fore-Bearing and Back-Bearing

$$\text{if FB} > 180^\circ,\ \text{BB} = \text{FB} + 180^\circ$$

$$\text{if FB} < 180^\circ,\ \text{BB} = \text{FB} - 180^\circ$$

$$\text{Included Angle, } \angle ABC = FB_{BC} - BB_{AB}$$

6.4 Instrument Description

6.4.1 Total Station

A total station is an electronic/optical instrument used in modern surveying and building construction that uses electronic transit theodolite in conjunction with electronic distance meter (EDM). It is also integrated with microprocessor, electronic data collector and storage system. The instrument is used to measure sloping distance of object to the instrument, horizontal angles and vertical angles. This Microprocessor unit enables for computation of data collected to further calculate the horizontal distance, coordinates of a point and reduced level of point.[4]

Table 6: Total Station Accuracy

Description	**Mode**	**Accuracy**
Angle	-	1"
Distance	Prism	Standard: 2mm+2ppm
		Tracking: 4 mm + 2 ppm
	DR Mode	Standard: 2mm+2ppm
		Tracking: 4 mm + 2 ppm

6.4.2 Reflector/Prism

Optical Survey prisms are a specially designed retro reflector, specifically a corner reflector, that is used to reflect the Electronic Distance Measurement (EDM) beam of a total station. The survey prism reflects the EDM beam back to its source with both a wide angle of incidence and with high precision. Distances can be determined by calculating the number (n) of wavelengths (λ) travelled.

$$D = \frac{n\lambda + \varphi}{2}$$

Figure 14: Trimble S5 Total Station

Figure 15: Prism

6.4.3 Bipod and Tripod

Total Station is mounted on tripod and fixed with a screw provided on the tripod, also the height of instrument can be adjusted by spreading the tripod legs.
Prism mounted on bipod provides stability to the prism, it comes with a pointed bottom which can be easily setup over a marked point and normality can be ensured by using the bubble tube provided on it.

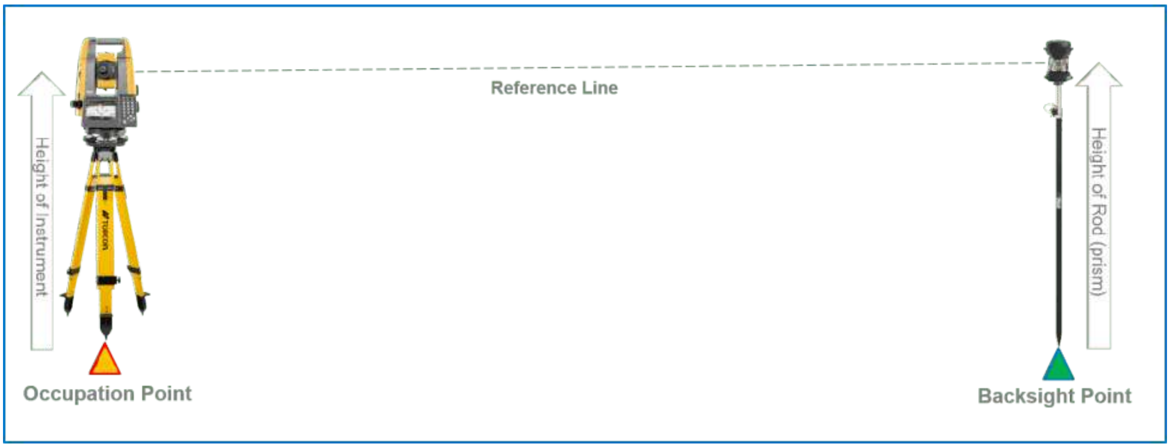


Figure 16: TS Setup

6.5 Procedure

1. Setting up tripod over the control point and roughly levelling it using a pen.

2. Centering was then carried out using the optical plummet.

3. Instrument was mounted over the tripod and fixed with the help of clamp provided with the tripod and levelling was done by centering the bubble with the three screws on tribach. Two screws for making the bubble on 'y' axis. For this we have to rotate both the screws either inward or outward at a time and the third screw to make bubble at center.

4. New job was created and electronic levelling was done with the screws on the tribach.

5. Bipod, prism and pole arrangement was setup over the backsight point.

6. In Measure Topo option the previous station with respect to instrument position was backsighted and distance was noted.

7. Instrument was rotated in clockwise direction and next immediate station was foresighted and distance and included angle was noted.

8. Instrument was shifted to next control station where foresight reading was taken.

9. Step 1 to Step 7 was repeated until all the included angles of the proposed traverse network was completed.

6.6 Observation Table

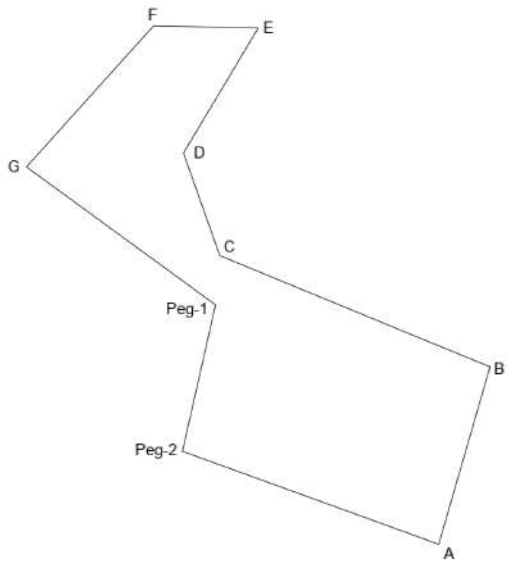


Figure 17: Traverse Net

Table 7: Observed Internal Angles

BS	Station Occupied	FS	Length (m)	Internal Angles		
				Degrees	Minutes	Seconds
D	E	F	47.207	60	5	1
E	F	G	85.1115	130	41	46
F	G	Peg-1	104.914	84	5	55
G	Peg-1	Peg-2	66.898	246	53	36
Peg-1	Peg-2	A	122.384	97	8	30
Peg-2	A	B	82.6915	85	55	59
A	B	C	131.0965	96	23	52
B	C	D	48.4985	228	4	27
C	D	E	65.496	230	40	15
			=754.297		**=1259°59'21"**	

6.7 Arithmetic Check and Quality of Work

According to the closing error, the quality of the survey work is defined. The order of survey and their respective permissible limits are mentioned below:

Table 8: Permissible Limits for Closing Error in Angle

Quality	Permissible Limit Formula	Permissible Limit (n=9)
First Order	$6''\sqrt{n}$	18"
Second Order	$15''\sqrt{n}$	45"
Third Order	$30''\sqrt{n}$	90"

The sum of total internal angle of a n-sided polygon is given by $(2n\text{-}4) \times 90°$. For 9 sides in proposed traverse sum of internal angles = 1260° which indicates an error in closing the traverse which can be computed as:

$$\text{Closing Error} = 1260° - 1259°59'21" = 0°0'39"$$

6.8 Adjustment using Bowditch Method

With closing error in angle there will be a closing error in distance also and it is given by the following formula where departure is given by $L\sin\vartheta$, $\vartheta = \text{FB}$, (L = length) and latitude is given by $L\cos\vartheta$:

$$\varepsilon = \sqrt{(\Sigma\,\text{Departure})^2 + (\Sigma\,\text{Latitude})^2}$$

Relative error is generally represented in 1 : X format and is given by:

$$\varepsilon_r = \frac{\varepsilon}{\text{Perimeter}}$$

Table 9: Permissible Limits for Closing Error in Distance

Quality	Permissible Limit (Relative Error)
First Order	1:25000
Second Order	1:10000
Third Order	1:5000

According to this method the errors in latitude and departure are distributed proportional to the length. Correction in Latitude/Departure = $\frac{\text{Length of line}}{\text{Perimeter of traverse}} \times$ (Error in Latitude/Departure)

6.8.1 Working with AutoCAD

Latitudes and Departures are calculated by following steps:

- New drawing file was created.
- With the traverse data using lines and angles traverse loop was plotted with line command.
- Line was drawn passing through the control station and point towards North-South direction.
- Similarly a transverse line was drawn passing through control station and pointing towards East-West direction.
- Using DAN (angular dimension) command FB of all lines was measured and following results were obtained from north direction.

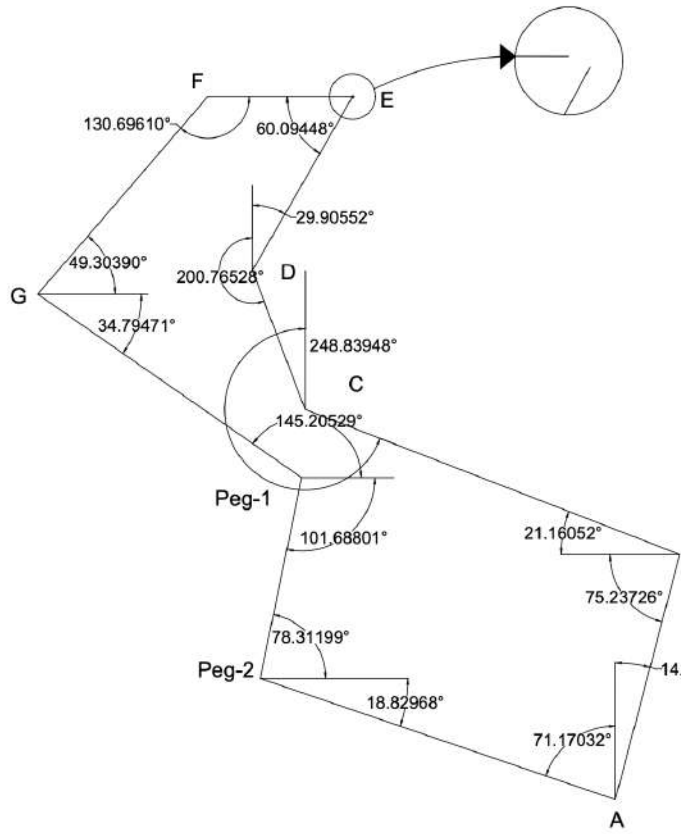


Figure 18: Traverse Plot in AutoCAD

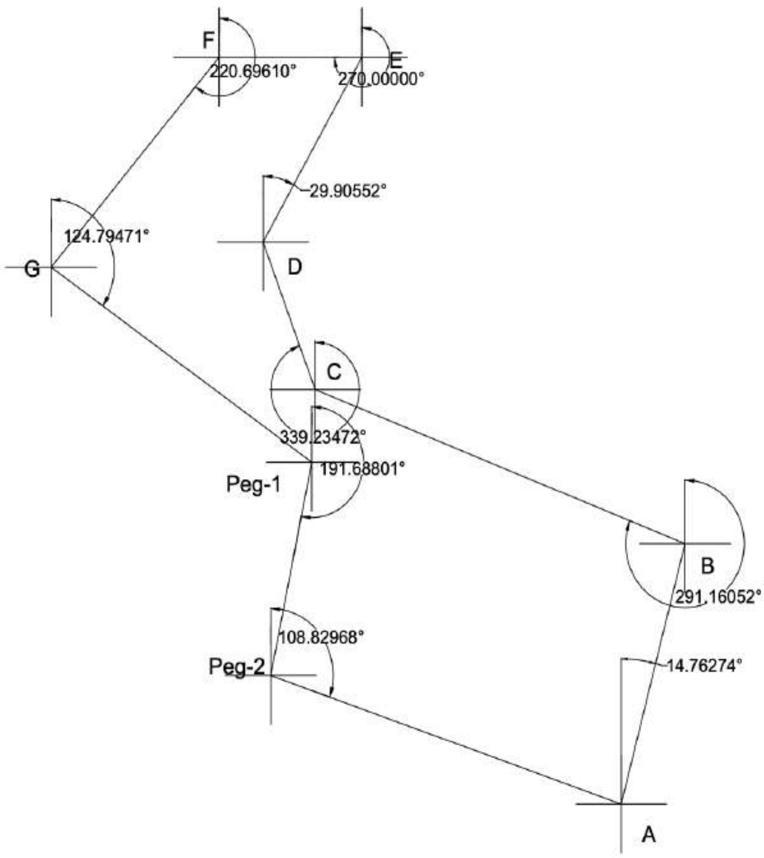


Figure 19: WCB Calculation

Table 10: Latitude and Departures

Station-line	**Length (m)**	**WCB (degrees)**	**Departure (m)**	**Latitude (m)**
F-G	85.1115	220.6916°	-55.4916127	-64.53408674
G-Peg1	104.914	124.79471°	86.15557597	-59.86788893
Peg1 - Peg2	66.898	191.68801°	-13.55235565	-65.51088505
Peg2 - A	122.384	108.82968°	115.834277	-39.50017389
A - B	82.6915	14.76274°	21.07119742	79.96179595
B - C	131.0965	291.16052°	-122.2570248	47.32348476
C - D	48.4985	339.23472°	-17.19467823	45.34807099
D - E	65.496	29.90552°	32.65442299	56.77512373
E-F	47.207	270°	-47.207	0
Total	**=754.297**		**=0.0128**	**=-0.00455**

Closing Error, $\varepsilon = \sqrt{(0.0128)^2 + (0.00455)^2} = 0.01358\,\text{m}$
Relative Error, $\varepsilon_r = \frac{1}{\frac{754.297}{0.01358}} = 1 : 55545$

Table 11: Adjusted Results with Bowditch Method

Station-line	length	WCB (deg)	Latitude (m)	Departure (m)	Latitude Correction	Departure Correction	Corrected Latitude (m)	Corrected Departure(m)	Corrected Length (m)	Corrected WCB
F-G	85.1115	220.6961	-64.5297	-55.4967	-2.26799E-05	-0.0009	-64.5298	-55.4976	85.1121	220.6965
G-Peg1	104.9140	124.7947	-59.8679	86.1556	-2.79568E-05	-0.0011	-59.8679	86.1545	104.9131	124.7951
Peg1 - Peg2	66.8980	191.6880	-65.5109	-13.5524	-1.78265E-05	-0.0007	-65.5109	-13.5530	66.8982	191.6886
Peg2 - A	122.3840	108.8297	-39.5002	115.8343	-3.26121E-05	-0.0013	-39.5002	115.8330	122.3828	108.8299
A - B	82.6915	14.7627	79.9618	21.0712	-2.20351E-05	-0.0008	79.9618	21.0703	82.6913	14.7622
B - C	131.0965	291.1605	47.3235	-122.2570	-3.49337E-05	-0.0013	47.3234	-122.2584	131.0977	291.1603
C - D	48.4985	339.2347	45.3481	-17.1947	-1.29236E-05	-0.0005	45.3481	-17.1952	48.4987	339.2342
D - E	65.4960	29.9055	56.7751	32.6544	-1.74529E-05	-0.0007	56.7751	32.6538	65.4957	29.9050
E-F	47.2070	270.0000	0.0000	-47.2070	-1.25794E-05	-0.0005	0.0000	-47.2075	47.2075	270.0000
Total	**=754.2970**		**=-0.0002**	**=0.0077**			**=-0.0004**	**=0.0000**	**=754.2970**	

6.9 Conclusion

From the obtained results it can be concluded that closing error in angle of **39"** lies in second order where as closing error in distance of **13.58 mm** lies in first order and can be accepted. However, using adjustment methods errors can be distributed over the network. Since the least count of total station in measuring angle is 1", this exercise can be repeated again for obtaining better results. The results obtained from traversing can be used in densifying control station by triangulation (creating a network of triangles) for defining the horizontal datum of the region.

7

CHAPTER

GNSS Survey

7.1 Introduction

Global Navigation Satellite Systems (GNSS) include constellations of Earth-orbiting satellites that broadcast their locations in space and time, of networks of ground control stations, and of receivers that calculate ground positions by trilateration. GNSS are used in all forms of transportation: space stations, aviation, maritime, rail, road and mass transit. Positioning, navigation and timing play a critical role in telecommunications, land surveying, law enforcement, emergency response, precision agriculture, mining, finance, scientific research and so on. They are used to control computer networks, air traffic, power grids and more.

7.2 Theory

- Each satellite in GNSS constellation transmits its ephemeris and the time at which signal left using atomic clocks mounted onboard.
- Receiver on the ground receives this signal and computes the travel time using atomic clocks. Therefore,

$$\text{Range, } r = ct$$

  where c = speed of light and t = time delay

- Since 'c' is the speed of light in vacuum and atomic clocks used in receiver is not as accurate as that of satellite, computed range needs to be corrected, so it is known as 'Pseudorange'.

$$\text{Pseudorange, } \rho = ct + ct_u$$

where t_u is called receiver clock bias.
(X_i, Y_i, Z_i) are the coordinates of the i^{th} satellite which is known
(X_u, Y_u, Z_u) are the coordinates of the receiver which is unknown

$$\rho_1 = \sqrt{(X_1 - X_u)^2 + (Y_1 - Y_u)^2 + (Z_1 - Z_u)^2} + ct_u$$

$$\rho_2 = \sqrt{(X_2 - X_u)^2 + (Y_2 - Y_u)^2 + (Z_2 - Z_u)^2} + ct_u$$

$$\rho_3 = \sqrt{(X_3 - X_u)^2 + (Y_3 - Y_u)^2 + (Z_3 - Z_u)^2} + ct_u$$

$$\rho_4 = \sqrt{(X_4 - X_u)^2 + (Y_4 - Y_u)^2 + (Z_4 - Z_u)^2} + ct_u$$

The above four equations has four unknowns and can be solved for X_u, Y_u, Z_u and t_u
Therefore, it takes minimum four GPS satellites to calculate a precise location on the Earth using the Global Positioning System: three to determine a position on the Earth, and one to adjust for the error in the receiver's clock. However for enhancing the accuracy, GNSS constellations have more than 4 satellites where solution can be obtained using least squares approach.

7.3 Instruments Description

- R10 Dual Frequency Receiver
- Controller
- Processing software: Trimble Business Center (TBC)

Figure 20: GNSS Rover Setup

7.4 Procedure

1. Receiver base was setup on the first day of the arrival.
2. Receiver (rover) was mounted on the bipod and setup was done at all the proposed control stations.
3. Using controller new job was created followed by selecting coordinate system as:
   Co-ordinate system: UTM
   Zone: 44 North
   Datum: WGS 84 (7P)
4. Since we wanted to collect data in rover mode with base providing the corrections, we chose fast-static mode in which data was collected for 30 minutes at every control station and 45 minutes in case of any obstruction.
5. Antenna type and height was chosen to be R10-2 and 2m respectively.
6. Under 'measure to' option parameters were set as follows:
   Logging interval: 15 seconds
   PDOP : 6.0
   Elevation Mask: 10°
7. Now we tried to connect the controller and the receiver to record observations. This connection was made via bluetooth and corresponding receiver serial number was selected.
8. Under 'start survey' option point was named as per the desciption and timing was set as per presence of obstruction.
9. Coordinates were stored automatically after elapsing of the input time and setup was then shifted to next control point. This was repeated for all control points.
10. The recorded observations were then imported to TBC software where baseline processing was done and corrections for the same epoch was applied.

7.5 Conclusion

The final report generated after post processing contains the point list with coordinates of all stations, residuals graph plot, etc. Spikes in residual plot indicates the presence of cycle slip or loss of lock which may occur due to obstruction in line of sight. GDOP and PDOP values represents dilution of precision (depends on geometry of satellites) must be within 5 and 3 respectively during observation and to avoid multipath effect, control station can be preferred away from water source.

7.6 Results

Table 12: Point List

ID	Easting (m)	Northing (m)	Elevation (m)
01122022-02	486201.153	2781806.798	86.533
day04	486201.153	2781806.798	86.533
Peg-1	486254.756	2781744.389	87.331
Peg-2	486239.868	2781679.186	86.456
A	486354.820	2781637.340	91.200
B	486377.504	2781716.827	90.650
C	486256.313	2781766.614	89.305
D	486240.070	2781812.302	90.096
F	486226.677	2781869.336	86.965
E	486273.844	2781868.372	89.520
G	486169.887	2781805.985	85.589

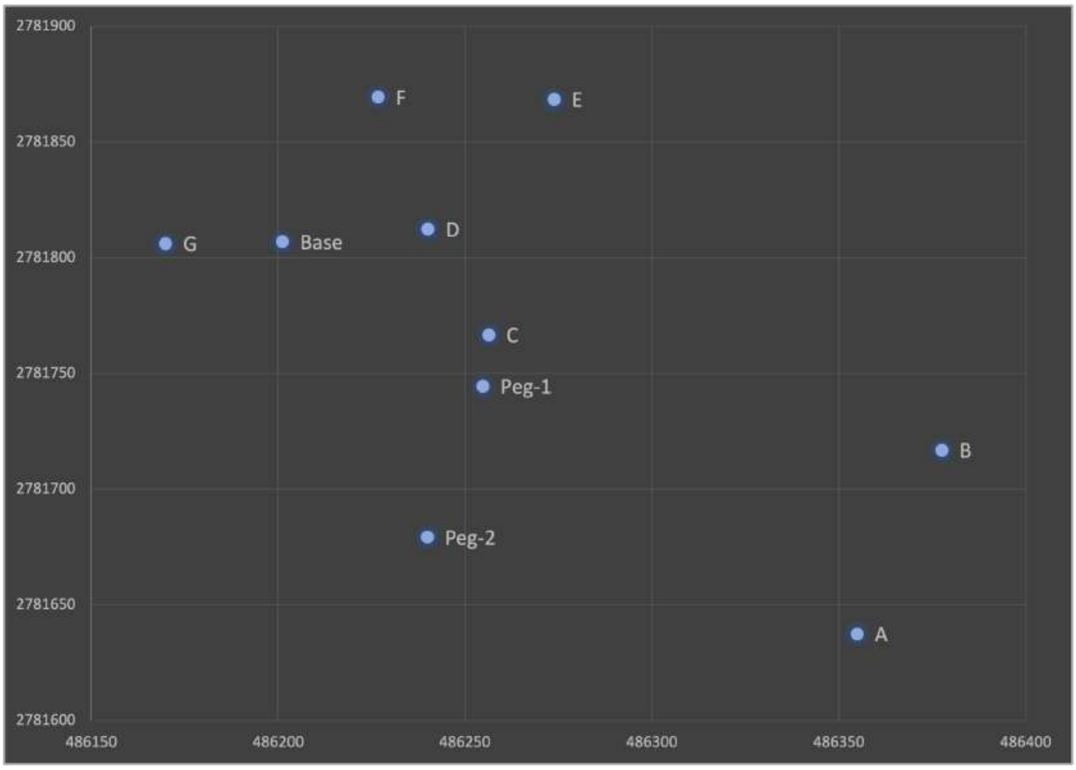


Figure 21: Scatter Plot of Control Station Coordinates

CHAPTER 8

Topographic Map and Road Profiling

8.1 Introduction

In modern mapping, a topographic map or topographic sheet is a type of map characterized by large-scale detail and quantitative representation of relief features, usually using contour lines (connecting points of equal elevation), but historically using a variety of methods. Traditional definitions require a topographic map to show both natural and artificial features.[1]

8.2 Objective

The aim of this exercise is to collect all types of man made and natural features from the control stations with known coordinates and orthometric heights.

8.3 Instrument Required

- Total Station
- Prism with Bipod
- Tripod
- Measuring Tape
- Consumable Items: Paper, Paint
- Software used: ArcGIS Pro

8.4 Procedure

8.4.1 Working with Total Station

Instrument Setup

- Setting up tripod over the control point and roughly levelling it using a pen.
- Centering was then carried out using the optical plummet.
- Instrument was mounted over the tripod and fixed with the help of clamp provided with the tripod and levelling was done by centering the bubble with the three screws on total station. Two screws for making the bubble on y axis. For this we have to rotate both the screws either inward or outward at a time and the third screw to make bubble at center.
- New job was created and electronic levelling was done with the screws on the tribach.

Feature Mapping

- In station set up option coordinates of control point and orthometric height along with instrument height was fed.
- For backsighting prism/reflector was setup at the other control station then the coordinates of that point with orthometric height and prism height was provided.
- Under Measure Topo option, a code for feature name was created and with prism at the point of interest corresponding coordinate was measured.
- Readings were taken by enabling the auto-lock feature by clicking on the prism icon on right hand side.
- DR mode of observations was selected in case the line of sight is obstructed. TS was focused directly to the point of interest and coordinates were obtained.
- This was done for all the features including line feature, polygon feature and point feature.
- For contouring, few points were taken on area having flat terrain and large number of points were taken where undulations were present.

Road Profiling

- Points were marked along the road at 5m interval longitudinally upto 300 m and total 5 points in cross sectional direction.
- Instrument was set up at the control point and using the above steps sighted to an intermediate point.
- Instrument was then shifted to a intermediate point and with prism at the control station backsighting was done to get the coordinates.
- Prism was kept at every marked point and readings were stored.

8.4.2 Working with ArcGIS Pro

Feature Mapping

- From total station data in the form of easting, northing, elevation, remarks etc. was transferred in csv format.
- In ArcGIS a new project was created.
- Using Display XY data, easting, northing and labels from csv file was selected and imported.
- In the same window projected coordinate system WGS 1984 44N was selected.
- For similar type of data label, a feature class was created to categorise data in groups using attribute query:
  Select by Attributes ⟶ Right click on label ⟶ New ⟶ Feature Class ⟶ Where = Class name ⟶ is equal to ⟶ data label
- Similarly all the labels were classified into appropriate classes.
- While collecting features such as buildings, all corner points were not visible from any station, therefore in order to correct this basemap was added to the layer and corner points were established and connected with smooth line using editor tool. Thus a polygon was created.
- Similarly for classes such as roads, gardens, water tanks, etc a polygon was created.
- After the previous step, appropriate symbology was chosen for each class by left clicking on feature class then selecting symbology option.
- Details such as contour interval, projection system, scale, title, names were then added and then exported to pdf in 600 dpi.

Contouring While making a map contour interval depends on the following factors:

1. Inversely proportional to scale of the map
2. Pupose of the map: for more detail, interval should be kept small.
3. Nature of ground

For generating contours, data was imported along with adjusted orthometric heights to the current project and the followed the below steps:

- In ArcGIS Pro, select either the Analysis or the Imagery tab. Click Raster Functions.
- In the Raster Functions pane, System > Surface > Contour.
- In contour properties, contour interval was selected and a new layer was created.
- In analysis tab ⟶ Geoprocessing pane input raster file and contour interval was selected and run.

8.5 Results

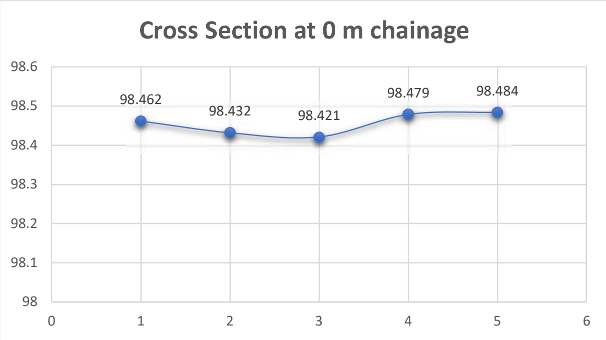
Topographic Map of Aarogyadham, Chitrakoot

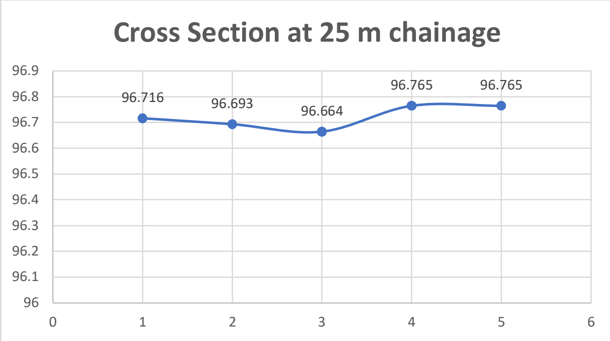
Submitted to: Prof Onkar Dikshit
Geoinformatics, dept. Civil Eng. IITK: CE673A

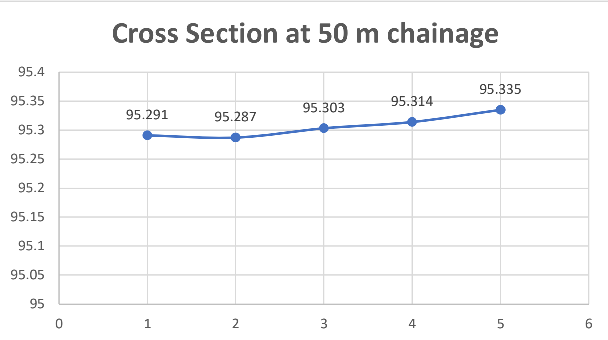
Assisted by:
1. Hari Babu Prajapati
2. Shitla Prasad Tripathi
3. Vipul Sharma & All TAs

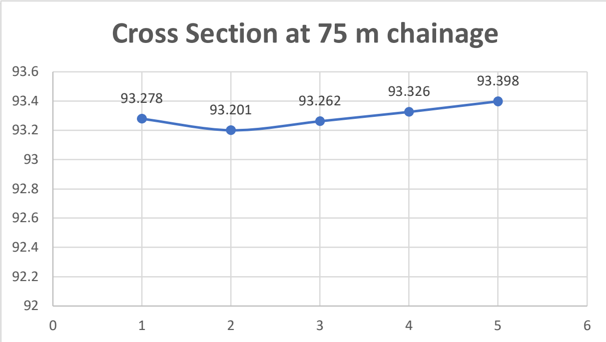
Submitted By:
1. Mintesnot Berhanu (22103272)
2. Subham Raj (22103070)
3. Brig. Sridhara Rao (22103087)
4. Abhishek Mhamane (22103401)

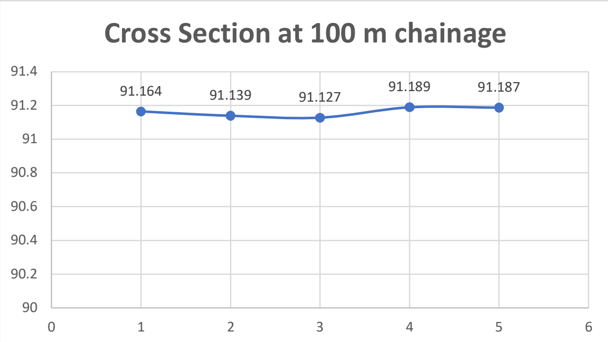
Road Profiling

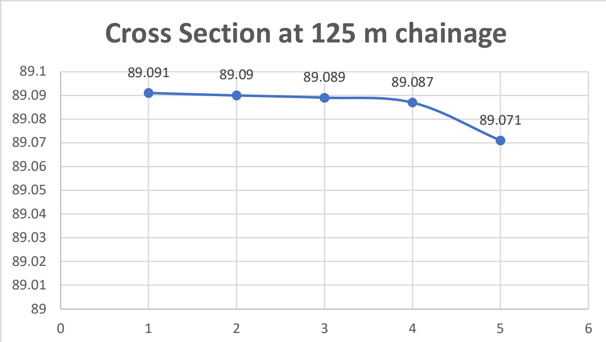


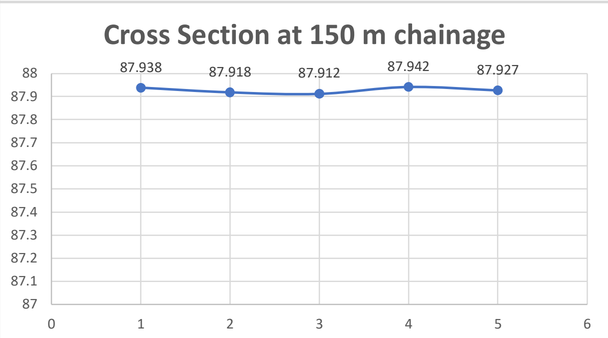


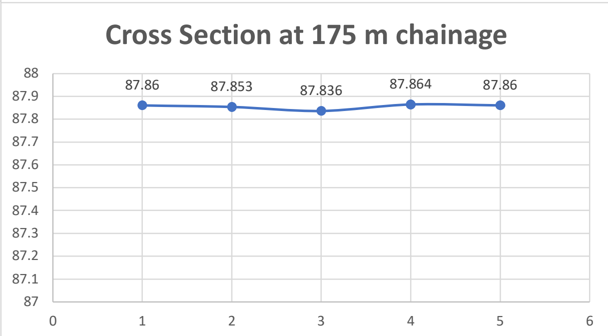


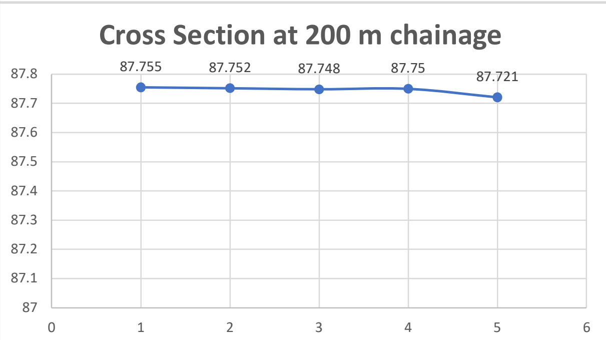


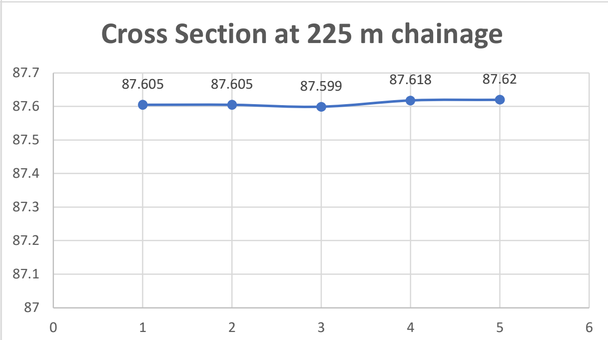


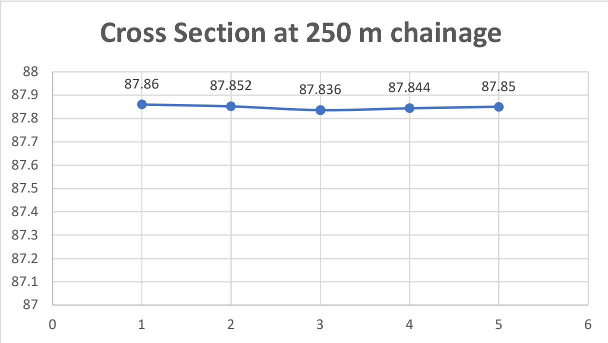











Cross Section at 250 m chainage
87.86
87.852
87.836
87.844
87.85
88
87.9
87.8
87.7
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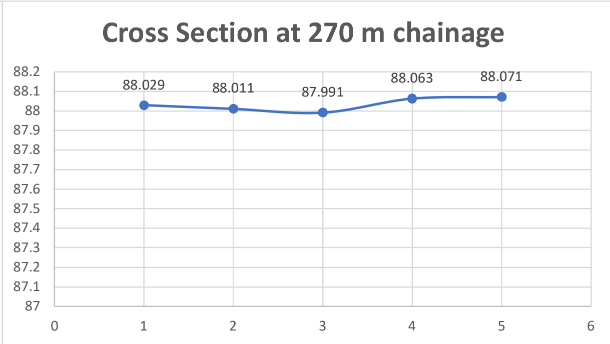
2
3
4
5
6


Cross Section at 270 m chainage
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88.011
87.991
88.063
88.071
88.2
88.1
88
87.9
87.8
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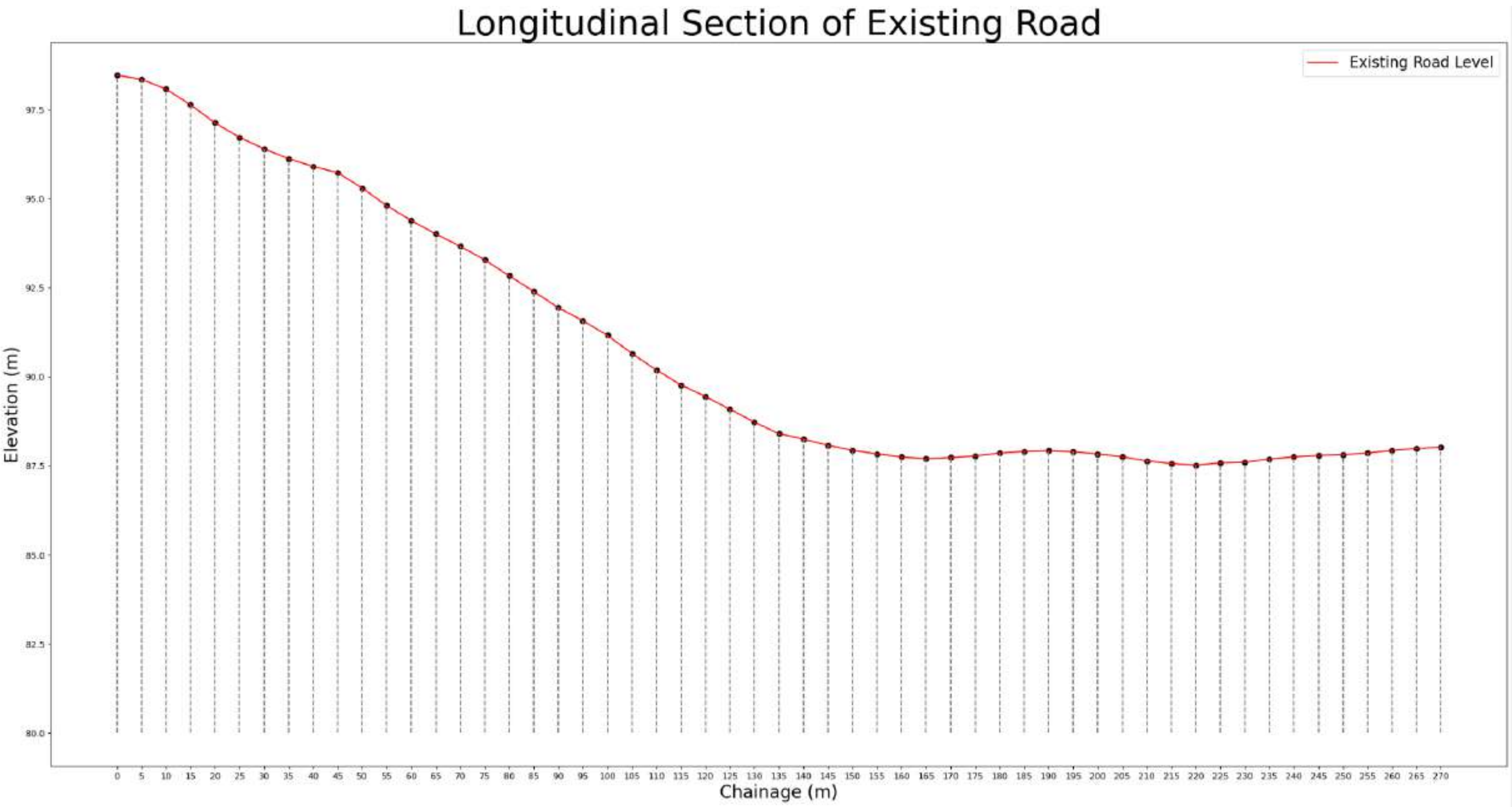
2
3
4
5
6


Longitudinal Section of Existing Road
Existing Road Level
Elevation (m)
97.5
95.0
92.5
90.0
87.5
85.0
82.5
80.0
Chainage (m)
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100 105 110 115 120 125 130 135 140 145 150 155 160 165 170 175 180 185 190 195 200 205 210 215 220 225 230 235 240 245 250 255 260 265 270

8.6 Conclusion

After feeding the backsight coordinates and sighting to the backsight point, there is an error between GNSS coordinates and Total Station computed coordinates, thereby affecting the accuracy of the map. There could be a few possible reasons for this discrepancy. Some possible reasons include:

- Instrument calibration issues
- Atmospheric conditions affecting the accuracy of Total Station measurements
- Human error
- Cycle Slip or signal interference in GNSS observations.

A threshold limit for the error can be set for controlling the error in propagation in map, exceeding which the observation will be discarded and repeated again.

Plottable Error in Map: Plottable error is the size of an object which will be represented as a point feature on the map, irrespective of shape and size. For proposed scale of 1:1500 plottable error = $1500 \times 0.25 = 37.5$cm

CHAPTER 9

References

[1] Alexander Kent. Topographic maps: methodological approaches for analyzing cartographic style. *Journal of Map & Geography Libraries*, 5(2):131–156, 2009.

[2] B. C. Punmia. What is levelling? - surveying amp; levelling, Mar 2019.

[3] Wilfred Schofield and Mark Breach. *Engineering surveying*. CRC Press, 2007.

[4] Vasanth Vidyakar. What is total station?, May 2014.