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SURVEY OF INDIA

HANDBOOK OF TOPOGRAPHY

CHAPTER III

(Corrected upto 2009)

CONTROL BY GPS & TRIANGULATION / TRILATERATION

C O N T E N T S

Sl. No.	Topics	Page Nos.
	SECTION 1 – GPS SURVEY	
1	Frame work Control	4
2.	GPS Surveys - Introduction	4
	i. Data Necessary for Commencing GPS Surveys	5
	ii. GCPs of First Phase and Second Phase	5
	iii. Planning & Reconnaissance	5
	iv. Planning for Network Design	6
	v. Recee on the Ground	7
	vi. GPS Observation	8
	vii. GPS Observation Principle	12
3.	Different GPS Surveying Techniques	13
4.	Field Observation and Processing	19
5.	Steps of Computations to get Co-ordinates on the Ellipsoid of interest	22
	i. Height Adjustment	22
	ii. Reduction of Observed Slope distances on Ellipsoid	23
	iii. Adjustment of Network	24
	iv. Computation of Latitude and Longitude	24
6.	Construction of GPS Pillars - Monumentation	27
	i. GCP Pillars for Phase I	27
	ii. GCP Pillars for Phase II	31
	iii. GPS Stations for subsequent Surveys	31
7.	Exercise for Field – Observational Process	32
	i. Mission Planning	32
	ii. Duration of Observation	33
	iii. Downloading the GPS data from Trimble Receiver	33
	iv. Conversion of RAW data into RINEX	34
	v. GPS Baseline Processing	34
	vi. Network Adjustment	37
	SECTION 2 – TRIANGULATION	
1.	Introduction	39
2.	Planning for Triangulation	40
	i. Data necessary for commencing Triangulation	40
	ii. Topographical Triangulation	40
3.	Planning and Reconnaissance	41
	i. Triangulation / Trilateration Programme	41
	ii. Lay-out	41
	iii. Size of Triangle and number of Points	42
	iv. Grazing rays	42
	v. Reconnaissance	42
	vi. Construction of station	43

4.	Errors in Triangulation	43
5.	Instrument	45
	i. The WILD Universal Theodolite – T2	45
	ii. Reading System	46
	iii. Adjustment of Theodolite	47
	iv. Adjustment of Telescope	48
6.	Instructions to Observer	50
7.	Signals	51
8.	Observations	51
	i. Method of Observing Horizontal Angle	52
	ii. Method of Observing Vertical Angle	53
	iii. Horizontal and Vertical Angle Book	54
9.	Computational Methods	57
	i. Adjustment of Triangular Error	57
	ii. Computation of Sides of Triangle	57
	iii. Computation of known side	57
	iv. Computation of co-ordinates of points	58
	v. Computation of heights	59

SECTION 1. – GPS SURVEY

1. FRAME WORK CONTROL -

The first essential requirement to produce an accurate map is to cover the whole area with a number of carefully determined points which will form a framework on which to base the ensuing survey of the physical detail in the area. Such a method prevents the accumulation of any system of errors outside those in the actual framework.

The accuracy of a survey thus depends ultimately on the accuracy of its framework or basis.
The basis of a survey consists of points fixed by one, or a combination of the following methods:-

(1) By Global Positioning System (GPS)
(2) Trilateration
(3)Triangulation
(4) Astronomical determinations.
(5) Traverse.

Of these, the soundest modern method for topographical operations is beyond all question a system of accurate GPS survey where by undue accumulation of error is precluded in the extension of the error in the internal details. This chapter contains only GPS Survey, trilateration and triangulation. The topics astronomical determination and traverse are beyond the scope of this book.

2. **GPS SURVEY : INTRODUCTION –**

GPS is based on a constellation of 24 satellites orbiting the earth at very high altitude any body could think of them or "man made stars" to replace the stars that we have traditionally used for navigation. The satellites are high enough that they can avoid the problems encountered by land based systems and they use technology accurate enough to give pinpoint positions any where in the world, 24 hours a day. Surveyors are using GPS to make measurements down to a centimetre accuracy. It uses satellites and computers to triangulate positions any where on earth. The position is calculated from distance measurements to satellites.

Since GPS is an all weather, real time continuously available, economic and very precise technique almost unlimited possibilities are opened up for its use in geodesy, surveying, navigation and related fields. Satellite geodesy provides observational and computational techniques which allow the solution of geodetic problems. GPS provide the technique to determine the precise global, regional and local three dimensional positions i.e. establishment of geodetic control, setting up of a completely new field of control points; densification of extension of existing control, improvement of the existing control and contribution to the determination of height above geoid can be achieved by use of GPS.

The GPS control points are provided in three phases. In first phase the Ground Control Points (GCP) are provided at a spacing of about 300 km as an inter–station distance. These stations are connected to GPS permanent stations maintained by Survey of India & other institutions. The stations of first phase are observed for 72 hours. The adjusted coordinates of these points will be utilized as primary GCP's for subsequent surveys.

In second phase of observations, Ground Control Points are provided at an inter-station distance of about 30 km. The stations of first phase are fixed as known points. The GCPs of second phase are adjusted with respect to the stations of first phase.

At third phase all other GPS measurements have to be connected to GCP's of 2nd phase for the subsequent surveys. One advantage when compared with classical techniques is that no systematic densifications are necessary.

The accuracy of the individual GPS stations with respect to neighboring station is ± 1cm. At first phase level, GPS network is installed with International Terrestrial Reference Frame (ITRF) / International Global Navigation Satellite System Service (IGS) sites

i. Data Necessary for commencing GPS survey -

For providing control by GPS, no extra data is required except the GCPs established in first phase or second phase. During observations to provide new control points, old GCPs must be connected with new work and the control points must be adjusted taking old GCP of first phase or second phase as fixed stations.

ii. GCP's of first phase or second phase -

The GCPs are available at an inter station spacing of about 30 km in the area. The area may have GCP of first phase i.e. inter station spacing of about 300 km. The descriptions and coordinates can be obtained from Director, Geodetic & Research Branch for computation of coordinate of new GCPs.

iii. Planning and Reconnaissance -

While surveying with GPS, there is no need to have inter-receiver visibility to measure a base line. GPS receives no signals between themselves, but from satellites orbiting the earth. Planning of network can be made as good as possible. The only constraint to receive these signals is having a clear view of the sky. Tall, dense trees are not desirable near a GPS station. Select station away from tall building and walls. Choose a station with no obstructions above an inclination angle of 15-20 degrees. High power television or microwave transmitters near the station may cause interferences. Try to locate GPS stations about 1 km away from them. If the control is needed near by such obstructing locations, the control can be made by EDM / Total station traverse starting from GPS station selected for frame work.

GPS satellite surveying is a three dimensional measurement system based on observations of the radio signals of the NAVSTAR Global positioning system. The GPS observations are processed to determine station positions in Cartesian coordinates (x,y,z) which can be converted to geodetic co-ordinates (latitude, longitude and height above reference ellipsoid). With adequate connections to vertical control network i.e. points of the known orthometric heights. Elevations can be computed for the points with unknown elevations.

There are two methods by which station positions can be determined:

1. Point Positioning
2. Relative Positioning

1. Point Positioning : At site station data is processed to determine three dimensional coordinates (x,y,z) referenced to WGS84 earth centered reference datum. The present accuracy for GPS point position determination ranges between 50 cm to 10 m depending upon the accuracy of the ephemerides and period of the observations.

2. Relative Positioning : Two or more GPS receivers receives signal simultaneously from the same set of satellites. These observations are processed to obtain the component of base line vectors between observed stations.

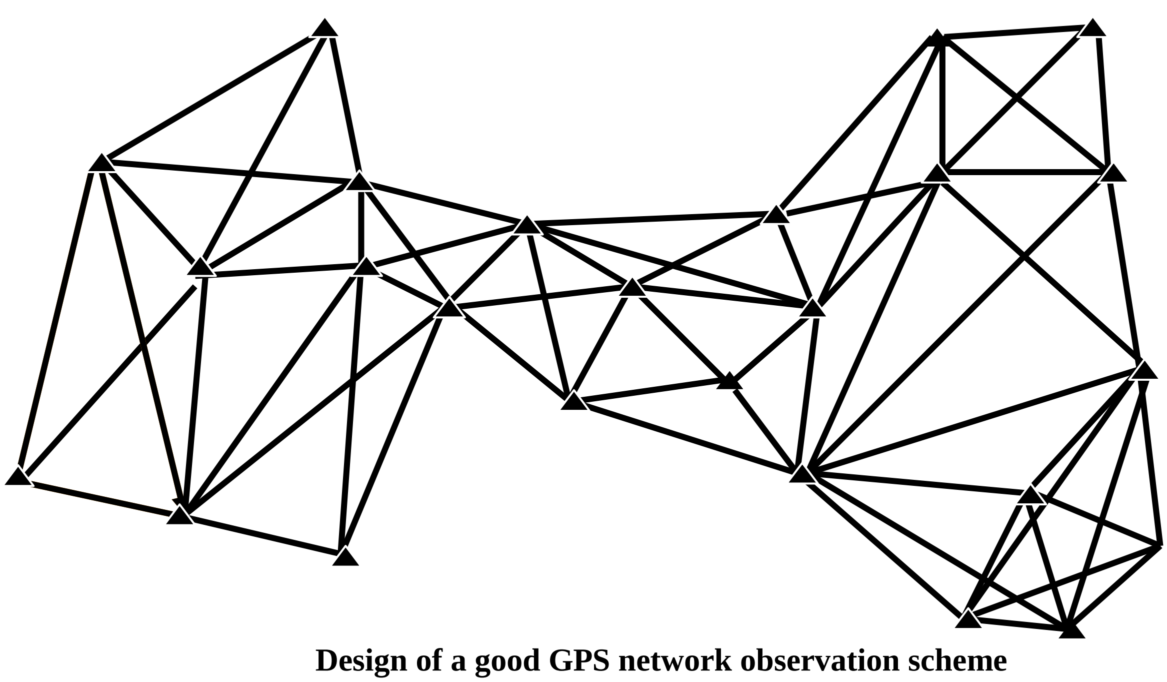
iv. Planning for Network Design -

General Information: GPS can be used like any other surveying tool; it can accomplish certain goals if we are conscious of its strengths and limitations. When surveying with GPS, we do not need to have inter-visibility between the stations to measure a baseline. The only constraint to receive the signals is having a clear view of the sky.

Network design on Map: To design of a GPS network, make a map of the station in a good geometric figure, both fixed control points and unknown points for the entire project area considering the every project structures. Correctly scale the map since distance between the stations is an important factor.

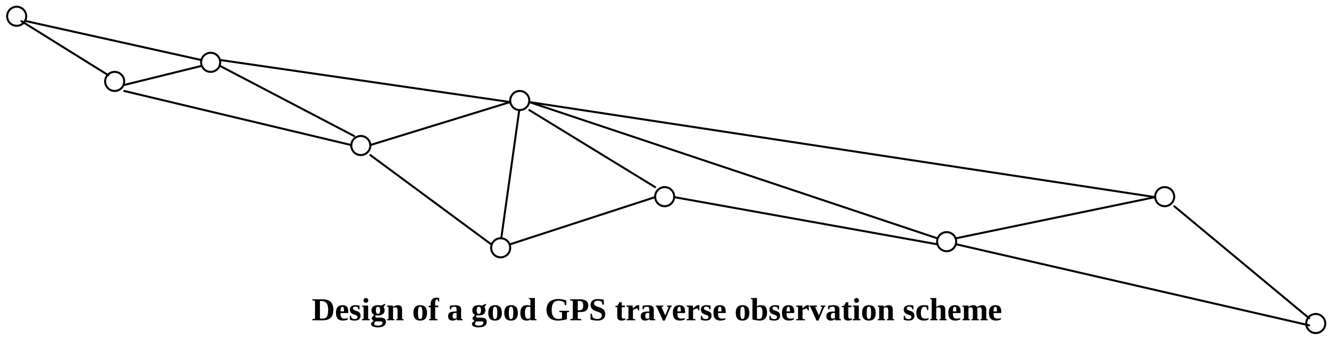
The confidence level of survey control points is dependent not only on the accuracy of the field measurements but also on the configuration of the control network.

For a network to fulfill its basic role as a strong and reliable reference framework, it must be homogeneous, feature a reasonable number of redundancies and the individual figures should be well-shaped. Stations should be as evenly spaced as possible, and all adjacent pairs of stations in the network should preferably be connected by direct measurement. The ratio of the longest length to the shortest should never be greater than one is to five and usually should be much less. Higher difference in heights between two stations will be avoided in the network. Network will be designed in such a way that it contains several small closed loops within the large network.



Design of a good GPS network observation scheme

The areas in which the ground does not permit to design the network, a GPS traverse required to be established as given below :



Design of a good GPS traverse observation scheme

There are two types of control points: 1. Horizontal Control 2. Vertical Control

Horizontal Control : A minimum of 3 to 4 fixed control points are needed in an average size of project for a complete adjustment. The more number of controls, the better the redundancy which results in a higher quantity of checks. Four control points, two at both end of the network are required to be established for network adjustment.

Vertical Control : MSL heights are not to be confused with GPS heights. This is because GPS heights are based on the WGS84 ellipsoid while MSL (Mean Sea Level) heights are based on an equipotential surface coinciding with MSL called the Geoid. To convert the GPS ellipsoidal heights to MSL heights we must need to know the geoidal undulation at that point i.e. Separation between Geoid and Ellipsoid.

As much as possible, maximum number of GPS stations of the network must be connected with conventional levelling procedures which can be used as fixed points for height adjustment. The height of the GPS Control Points which is not connected with levelling line will be adjusted by adjustment program with the help of fixed vertical control points and EGM96 Geoid model; **it will force the unknown vertical points onto the same datum as the fixed points.**

v. Recee on the ground –

All location of control points on the map will be established on the ground as possible so that design of the network will not be distorted and keeping the following points into the considerations:

1. Use of good geometry in the network.
2. Inter-visibility between the connecting points will be ensured at each feature of site.
3. Grazing ray will be avoided.
4. Clear visibility of sky above 15° of altitude all around the site is to be ensured.
5. No points should be selected near high tension power lines and other structures causing multipath / magnetic effect.
6. P.T. / Chart on scale 1:25,000 showing the location of new recceed stations with geometrical figure and location of standard bench marks will be prepared. Every Control point will be allotted a unique station ID number with name.
7. Description of each station and bench mark will be written as per the existing specification for G.T. stations / Standard bench marks.
8. Sketches with obstructions / curtains will be prepared for each station separately.

vi. GPS Observation –

A session is defined as where two or more receivers collect data simultaneously. The time of collecting data for a session is dependent on several factors, one being satellite availability. The time to end a session is dependent on how long we wish to collect simultaneous data. The length of data collection is related to, but not limited to, the distance between receivers.

Non simultaneous data collection is unproductive. Therefore it is important that receivers start collecting data for a specified time simultaneously. Time of observation should be selected by the Mission Planning Software.

The Global Positioning System consists of three segments :

1. The Space Segment
2. The Control Segment
3. The User Segment

The Space Segment ::

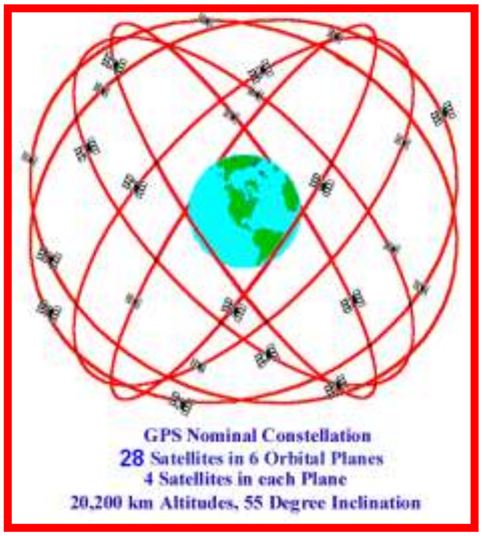
The Space Segment consists of constellation of space craft, and the signals that are broadcast by them, which allow user to determine position, velocity and time. The basic functions of the satellite are to:

- Receive and store data uploaded by control segment
- Maintain accurate time by means of onboard atomic clocks, and
- Transmit information and signal to user on two L-band frequency.

Several constellations of GPS satellite have been deployed, and are planned. The first experimental satellite of so-called 'Block I' constellation was launched in February 1978. The last of this 11 satellite was launched in 1985. The operational constellation of GPS satellites, the 'Block II' and 'Block IIA' satellite, were launched from 1989 onwards. Full operational capability was declared on 17 July, 1995 - the milestone reached when 24 'Block II/IIA' satellites were operating satisfactorily. There are 18 replenishment 'Block IIR' satellites, with the first launched in 1997. Currently 12 of these satellites are redesigned as part of the 'GPS modernization' program. The 'Block IIF' follow on satellite series is still in the design phase and the satellites are planned for launch from 2006 onwards with the similar enhancement as the latter 'Block IIR' satellite, as well as having the ability to transmit a third frequency.

At an altitude of approximately 20,200 km, a constellation of 24 functioning GPS satellite, located in six orbital planes inclined at about 55° to the equator, is sufficient to ensure that there will be at least four satellites visible, at any unobstructed site on earth, at any time of the day. As the GPS satellites are in nearly circular orbits.

- Their orbital period is nearly 11h 58min., so that each satellite makes two revolutions in one sidereal day (the period for the earth to complete one rotation about its axis with respect to stars).
- At the end of the sidereal day the satellites are again over the same location on the earth.
- Reckoned in terms of solar day (24 h in length), the satellites are in the same position in sky in about 4 min. each day.



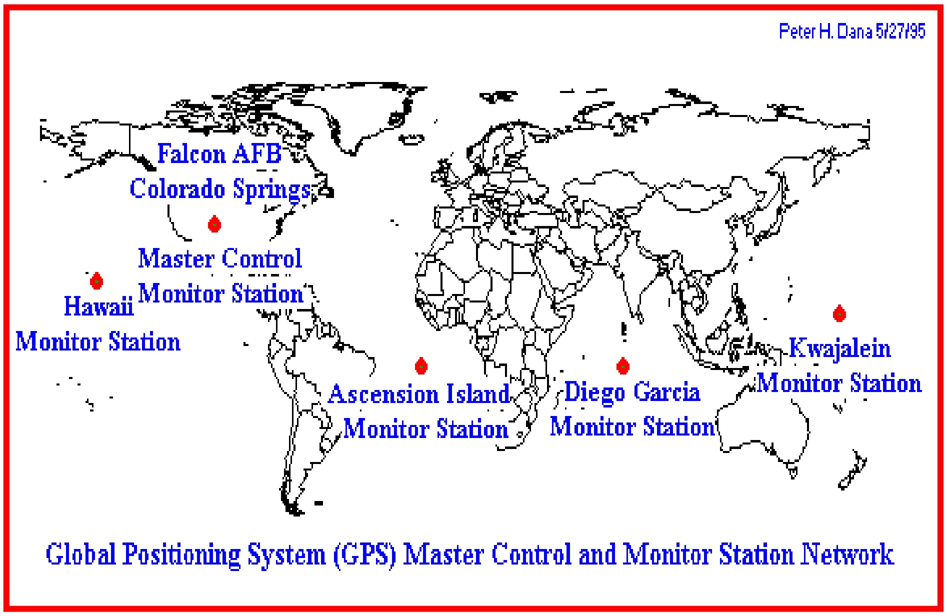
Satellite visibility at any point on the earth and for any period, can be computed using 'mission planning' tools provided with Standard GPS Surveying Software. A GPS Satellite may be above an observer's horizon for many hours, perhaps 6-7 hours or more in one pass. At various time of the day, and various locations on the surface of earth, the number of satellites and length of the time they are above an observer's horizon will vary. Although at certain times of the day these may be as many as 12 satellites visible simultaneously, there are nevertheless occasional periods of degraded satellite coverage. 'Degrade satellite coverage' is typically defined in terms of the magnitude of the Dilution of Precision (DOP) factor, a measure of the quality of satellite receiver geometry. The higher is DOP value, the poorer the satellite geometry. Each GPS satellite transmits unique navigational signal centered on two L-band frequencies of electromagnetic spectrum. L_1 at 1575.42 MHz and L_2 at 1227.60 MHz (Two signals at different frequencies permits the ionospheric delay effects on the signal ray paths to be estimated, thus improving measurement accuracy). At these two frequencies the signals are highly directional and can be reflected or blocked by solid objects. Clouds are easily penetrated, but signals may be blocked by foliage. The satellite signal consists of the following components:

- Two L-band carrier waves
- Ranging code modulated on the carrier waves
- Navigation message

The primary function of the ranging code is to permit the signal transit time (from satellite to receiver) to be determined. The transit time when multiplied by the speed of light then gives a measure of the receiver - satellite 'range'. In reality the measurement process is more complex and the measurement is contaminated by a variety of biases and errors. The navigation message contains the satellite orbit (or ephemeris) information, satellite clock error parameters and pertinent general system information necessary for real time navigation to be performed.

The Control Segment :

The control segment consists of facilities necessary for satellite health monitoring, telemetry tracking, command and control and satellite orbit and clock error computations. There are currently five ground facilities stations: Hawaii, Colorado Springs, Ascension Island, Diego Garcia and Kwajalein. All are operated by US Department of Defense and perform the following functions:



- All the five stations are monitor stations, equipped with GPS receiver to track the satellites. The resultant tracking data is sent to Master Control Station (MCS).
- Colorado Spring is the MCS, where tracking data are processed in order to compute the satellite ephemeredes (or coordinate) and satellite clock error parameters. It is also the station that initiates all operations of Space Segment, such as space craft manoeuvering, signal encryption, satellite clock-keeping etc.
- Three of the stations (Ascension Island, Diego Garcia and Kwajalein) are upload stations through which data is telemetered to the satellite.

Each of the upload stations views all the satellites at least once per day. All the satellites are therefore in contact with an upload station several times a day, and now navigation messages as well as command telemetry can be transmitted to the GPS satellite on a regular basis. The computations of each satellite's ephemeris, and the determination of each satellite's clock errors, are most important tasks of Control Segment. The first is necessary because the GPS satellite functions as 'orbiting control stations' and their coordinates must be known to a relatively high accuracy, while the latter a significant measurement bias to be reduced.

The product of the orbit computation process at the MCS is each satellite's predicted ephemeris, expressed in the reference system most appropriate for positioning: Earth centered Earth - Fixed (ECEF) reference system known as World Geodetic System (WGS 84). The accuracy with which the orbit is predicted is typically at the few meter level. The behaviors of each GPS satellite clock is monitored against GPS time, as maintained by an ensemble of the atomic clocks at the MCS. The satellite clock biases drift and drift rate relative to GPS time are explicitly determined at the same time as estimation of satellite ephemeris. The clock error behaviors so determined is made available to all GPS users via clock error coefficients in a polynomial form broadcast is the navigation message. However, what is available to users is instant. Due to random deviations - even cesium and rubidium oscillators are not entirely predictable - the deterministic models of satellite clock error are only accurate to about 10 nano seconds or so. This is not precise enough for range measurements that must satisfy the requirements of Cm level GPS positioning. Strategies have therefore to be implemented that will account for this residual range bias.

The User Segment :

Appropriate satellite receivers are required to use the GPS signal for navigation purpose or for geodetic positioning.

Main components of GPS Receivers are as under:

- Antenna with pre-amplifier
- Precision oscillator (clock) - quartz
- Power supply - Ni - Cd 12 v Battery
- User interface - command and display panel
- Memory, data storage


The Antenna detects the electromagnetic waves arriving from satellite, converts the wave energy into electric current, amplifies the signal strength and hands the signal over to receiver electronics

Several antennas are available

- monopole or dipole
- helix
- spiral helix
- micro-strip
- choke ring

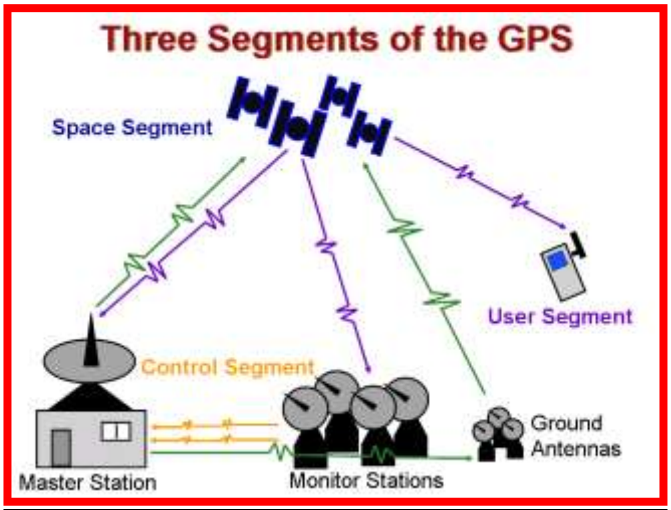


GPS receivers are classified based on the code and carrier phase available with them.

- C/A code
- C/A code + L_1 carrier phase
- C/A code + L_1 carrier phase + L_2 carrier phase
- C/A code + P code + L_1 L_2 carrier phase

Another classification of GPS receiver is possible with respect to user community.

- Military Receiver
- Civilian Receiver
- Navigation Receiver
- Geodetic Receiver



vii. GPS Observation Principle -

Let the signal transmit time form the satellite be T_t

Signal received time by the receiver is T_r

Synchronization error between receiver and satellite clocks is Δt

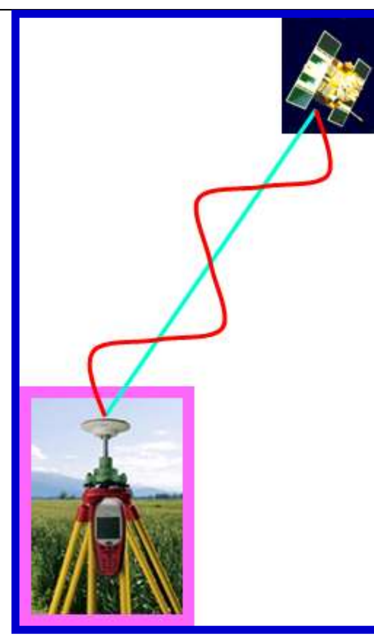
Velocity of EM signals = velocity of light = c

Pseudo Range (Between Receiver's Antenna and Satellite) is

$$R_i = \sqrt{(X_i - X_0)^2 + (Y_i - Y_0)^2 + (Z_i - Z_0)^2} - c.\Delta t$$

In the above equations (X_i, Y_i, Z_i) are the satellite coordinates known to the user from the broadcast ephemeris.

(X_0, Y_0, Z_0) are Antenna coordinate in which the user is interested that is an unknown and Δt the synchronization error between receiver and satellite clock is also unknown. To solve these four unknown the user requires four satellites to make four equations and can find (X_0, Y_0, Z_0) and Δt. The above equation require linearization to solve it



GPS has numerous advantages over traditional surveying methods:

1. Inter-station visibility between points is not required.

2. Can be used at any time of the day or night and in any weather.

3. Produces results with very high geodetic accuracy.

4. More work can be accomplished in less time with fewer people.

Limitations

In order to operate with GPS it is important that the GPS Antenna has a clear view to at least 4 satellites. Sometimes, the satellite signals can be blocked by tall buildings, trees etc. Hence, GPS cannot be used indoors. It is also difficult to use GPS in town centers or woodland.

Due to this limitation, it may prove more cost effective in some survey applications to use an optical total station or to combine use of such an instrument with GPS

3. DIFFERENT GPS SURVEYING TECHNIQUES -

There are several measuring techniques that can be used by most GPS Survey Receivers. The surveyor should choose the appropriate technique for the application.

Static - Used for long lines, geodetic networks, tectonic plate studies etc. Offers high accuracy over long distances but is comparatively slow.

Rapid Static - Used for establishing local control networks, Network densification etc. Offers high accuracy on baselines up to about 20km and is much faster than the Static technique.

Kinematic - Used for detail surveys and measuring many points in quick succession. Very efficient way of measuring many points that are close together. However, if there are obstructions to the sky such as bridges, trees, tall buildings etc., and less than 4 satellites are tracked, the equipment must be reinitialized which can take 5-10 minutes. A processing technique known as On the- Fly (OTF) has gone a long way to minimise this restriction.

RTK - Real Time Kinematic uses a radio data link to transmit satellite data from the Reference to the Rover. This enables coordinates to be calculated and displayed in real time, as the survey is being carried out. Used for similar applications as Kinematic. A very effective way for measuring detail as results are presented as work is carried out.

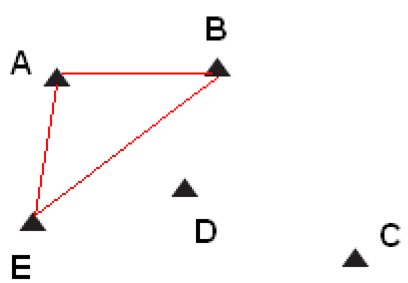
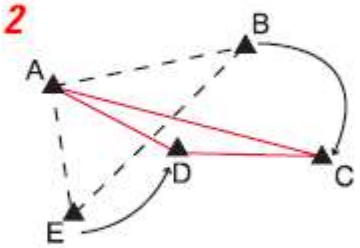
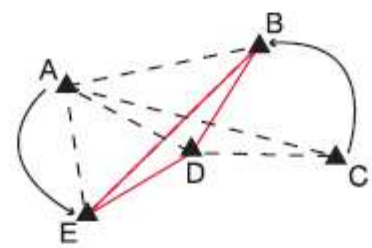
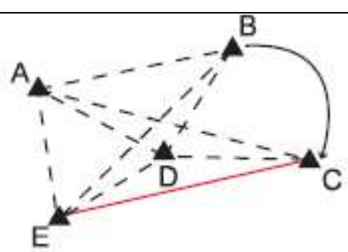
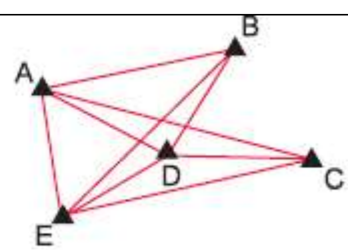
This technique is however reliant upon a radio link, which is subject to interference from other radio sources and also line of sight blockage. There are several measuring techniques that can be used by most GPS Survey Receivers. The surveyor should choose the appropriate technique for the application.

Static Surveys

This was the first method to be developed for GPS surveying. It can be used for measuring long baselines (usually 20km (16 miles) and over). One receiver is placed on a point whose coordinates are known accurately in WGS84. This is known as the Reference Receiver. The other receiver is placed on the other end of the baseline and is known as the Rover. Data is then recorded at both stations simultaneously. It is important that data is being recorded at the same rate at each station. The data collection rate may be typically set to 15, 30 or 60 seconds.

The receivers have to collect data for a certain length of time. This time is influenced by the length of the line, the number of satellites observed and the satellite geometry (dilution of precision or DOP). As a rule of thumb, the observation time is a minimum of 1 hour for a 20km line with 5 satellites and a prevailing GDOP of 8. Longer lines require longer observation times.

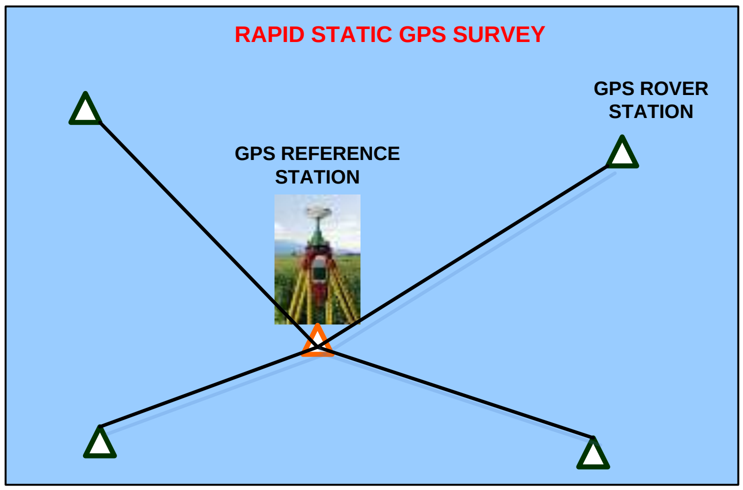
Once enough data has been collected, the receivers can be switched off. The Rover can then be moved to the next baseline and measurement can once again commence. It is very important to introduce redundancy into the network that is being measured. This involves measuring points at least twice and creates safety checks against problems that would otherwise go undetected. A great increase in productivity can be realized with the addition of an extra Rover receiver. Good coordination is required between the survey crews in order to maximize the potential of having three receivers. An example is given below.

1  The network ABCDE has to be measured with three receivers. The Coordinates of A are known in WGS84. The receivers are placed on A, B and E. GPS data is recorded for the required length of time.	2  After the required length of time, the receiver that was at E moves to D and B moves to C. The triangle ACD is measured.	3  Then A moves to E and C moves to B. The triangle BDE is measured.
4  Finally, B moves back to C and the line EC is measured.	5  	

Rapid Static Survey :

In Rapid Static surveys, a Reference Point is chosen and one or more Rovers operate with respect to it. Typically, Rapid Static is used for densifying existing networks, establishing control etc.

When starting work in an area where no GPS surveying has previously taken place, the first task is to observe a number of points, whose coordinates are accurately known in the local system. This will enable a transformation to be calculated and all hence, points measured with GPS in that area can be easily converted into the local system. At least 4 known points on the perimeter of the area of interest should be observed. The transformation calculated will then be valid for the area enclosed by those points.



The Reference Receiver is usually set up at a known point and can be included in the calculations of the transformation parameters. If no known point is available, it can be set up anywhere within the network. The Rover receiver(s) then visit each of the known points. The length of time that the Rovers must observe for at each point is related to the baseline length from the Reference and the GDOP.

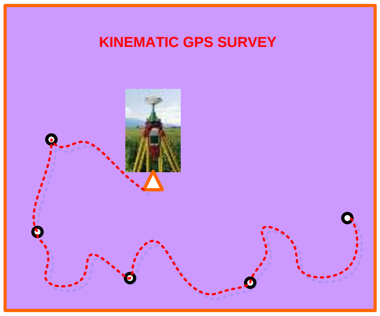
The data is recorded and post-processed back at the office. Checks should then be carried out to ensure that no gross errors exist in the measurements. This can be done by measuring the points again at a different time of the day.

When working with two or more Rover receivers, an alternative is to ensure that all rovers operate at each occupied point simultaneously. Thus allows data from each station to be used as either Reference or Rover during post processing and is the most efficient way to work, but also the most difficult to synchronize. Another way to build in redundancy is to set up two reference stations, and use one rover to occupy the points.

Kinematic Survey :

The Kinematic technique is typically used for detail surveying, recording trajectories etc., although with the advent of RTK its popularity is diminishing. The technique involves a moving Rover whose position can be calculated relative to the Reference.

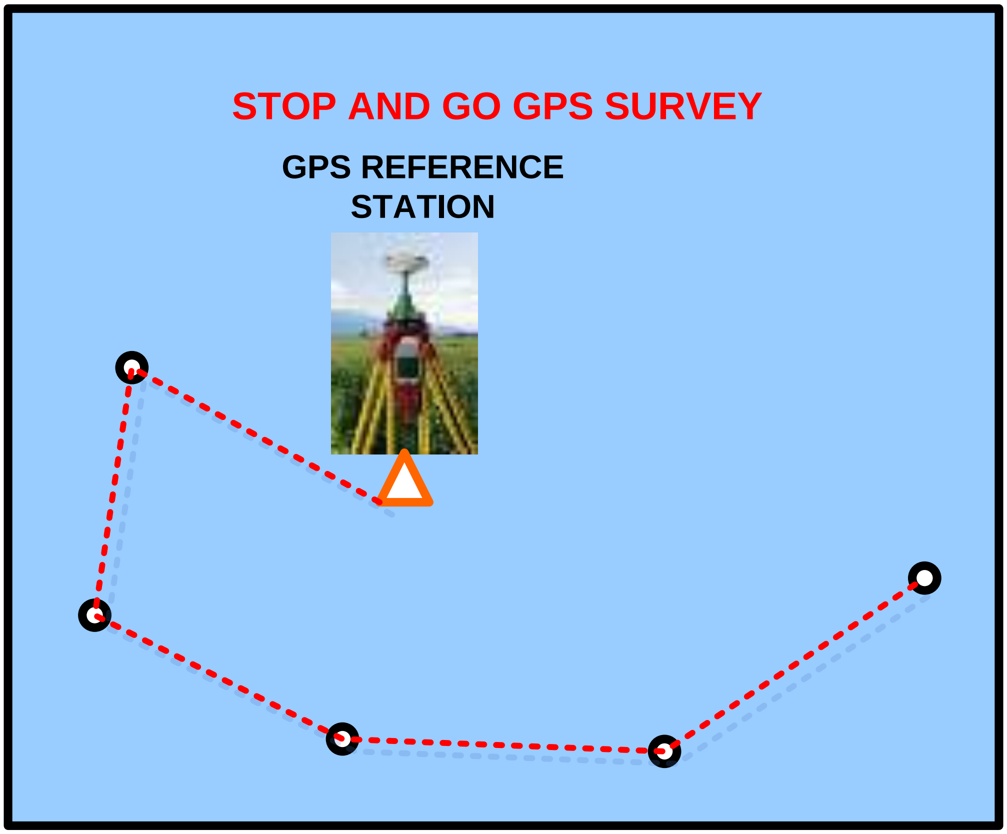
Firstly, the Rover has to perform what is known as an initialization. This is essentially the same as measuring a Rapid Static point and enables the post processing software to resolve the ambiguity when back in the office. The Reference and Rover are switched on and remain absolutely stationary for 5-20 minutes, collecting data. (The actual time depends on the baseline length from the Reference and the number of satellites observed).



After this period, the Rover may then move freely. The user can record positions at a predefined recording rate, can record distinct positions, or record a combination of the two. This part of the measurement is commonly called the kinematic chain. A major point to watch during kinematic surveys is to avoid moving too close to objects that could block the satellite signal from the Rover receiver. If at any time, less than four satellites are tracked by the Rover receiver, you must stop, move into a position where 4 or more satellites are tracked and perform an initialization again before continuing. Initialization is performed from the Reference to the Rover. The Rover can then move. Positions can be recorded at a predefined interval... ...and also at distinct points if required.

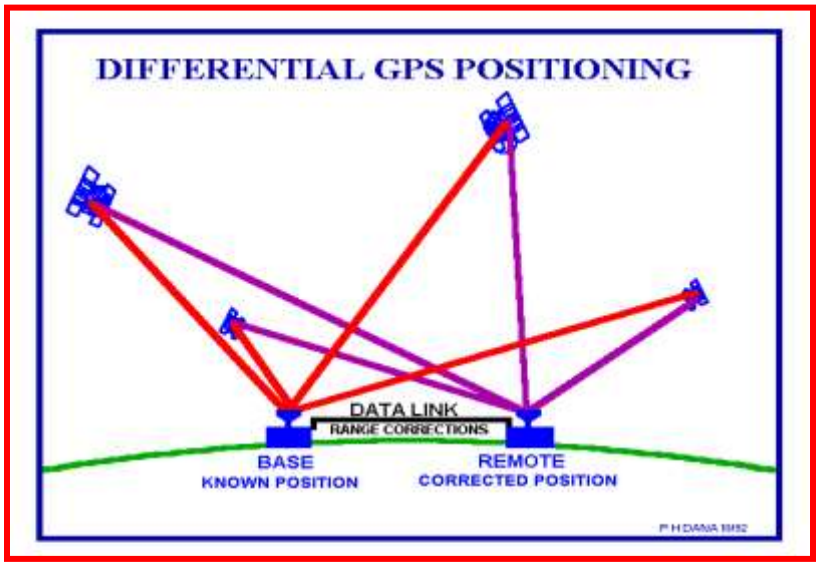
`Stop-and-Go' GPS Surveying :

This is the kinematic technique because the user's receiver continues to track satellites while it is in motion. It is known as stop-and-go (or semi-kinematic) technique because the coordinates of the receiver are only of interest when it is stationary (the stop part) but the receiver continues to function while is being moved (the `go' part) from one stationary set up to the next.



The DGPS Techniques :

In its simplest form, a DGPS reference receiver is set up at a site with known coordinates. After it has been configured to operate as the `base station', the reference receiver tracks continuously all the visible satellites, and determines the corrections necessary for its pseudo range data to be able to compute the SPP result that is identical to the known coordinates of the site. This correction data is then transmitted the user (via some form of wireless data link). The receiver then applies the corrections to their pseudo range data before computing these SPP solutions.



The RTK-GPS technique :

The RTK-GPS is an attractive technique for many survey applications as there is no post processing of the carrier phase data. The standard scenario requires the surveyor to operate two GPS receivers (one ***reference receiver*** and other ***`roving receiver'***), as well as the data link. Once set up, the reference receiver will continuously transmit its carrier phase measurements to the roving receiver. The software within `roving' receiver's microprocessor then tries to resolve the ambiguities in shortest time possible (using OTF-AR algorithm), and resulting **`carrier-range'** data is used to derive cm-level accuracy baseline results. These results may be stored (for later down loading) displayed and used for surveying applications or processed by a computer to guide control machinery such as excavators etc.

Successful operation of RTK system is usually limited to baseline lengths of 5-10Km because this is typically the inter receiver distance over which very rapid OTF-AR algorithm work reliably. As with the post-processed modes of carrier phase-based positioning, when signals are obstructed then the OTF-AR algorithm has to be started again in order to resolve the (new) ambiguities. As this may take several tens of seconds, and if signal interruptions occur frequently, then this `**dead time**' can result in RTK being a grossly inefficient precise positioning technique. Note that RTK need not only be used for purely kinematic applications. RTK equipment can be used in the stop-and-go or rapid-static mode of surveying as well, the crucial difference (and perhaps important advantage) being that the results are available immediately after data collection is completed.

RTK is especially vulnerable to poor satellite visibility, multi path and unreliable data link from reference station. This link is invariably a VHF or UHF wireless connection between reference and rover receivers. Unfortunately the data link tends to be less robust than, for example, data links to commercial DGPS procedures. The reasons typically are low power of signal, interference by other transmissions, and signal obstructions. Although proprietary data transmission formats are the norm for RTK operations, the RTCM format does provide message type that can be used for RTK.

4. FIELD OBSERVATION AND PROCESSING -

Planning of GPS observation - Before starting a GPS Survey Campaign, the following points should be kept in mind.

Pre Survey planning:-

- a satellite “sky plot” or visibility chart should be prepared for each site.
- Observation time is decided when the satellite visibility is maximum and PDOP is minimum.

Selection of site:- The possible location of GPS control points to be established are selected on map. Starting and closing GT station are also selected. While selecting the station the following points should be kept in mind:

- Distance between two stations should not be large and should be kept within 50km. As far as possible.
- Since inter-visibility between the stations is not required, the station with easier approach should be selected for speed and economy.
- A 15degree clearance from horizon to sky should be available. Where 15 degree clearance is not possible due to fixed location of points, GPS mission planning should be done in the office to select the right time of observation, to ensure maximum satellite visibility.
- No radio signal transmitter should be operational in the vicinity of the GPS point.
- The point selected should be such that it is not likely to be destroyed due to human activities like construction etc. In case the requirement of control points is of the temporary nature like for model control etc. few points at sufficient density should be selected which can be used as permanent control points for mapping and these should be constructed as per standard of GT station.
- Stations near H. T. power lines should be avoided.

Field Procedure - The GPS observation will yield coordinates of the survey stations in the World Geodetic System WGS-84, which is a geocentric datum. These coordinates are not very precise and accurate due to effects of Selective Availability (SA) and Anti-spoofing (AS).

However for precise work, GPS can be very effectively used in static relative mode (translocation mode), by using two or more receivers observing simultaneously at different locations. In this mode due to cancellation of major part of SA & AS effect the base lines can be estimated very accurately. This is the method being employed for all geodetic and topographical surveying with GPS in Survey of India.

The basic observable in the planimetric GPS observation is the base line vector between stations. If the co-ordinates are required in WGS-84 datum we need do much computation work. But,to derive the coordinates on Indian Geodetic Datum – the Everest Ellipsoid, the observations are started and closed on existing Geodetic Stations of the Great Trigonometrical (GT) triangulation network, whose coordinates on the Everest Ellipsoid are known.

The distances observed with GPS observation (after data processing we get slope distance) are projected on the Everest Ellipsoid using **computed ellipsoidal heights** and final network adjustment is carried out using the coordinates of known stations and the GPS derived base line vectors projected on Everest ellipsoid. For computation of height Bench Marks of existing H.P. / Precision / Secondary or tertiary levelling are connected with GPS network at regular interval.

In case of vertical control, the basic observable is the MSL heights (orthometric heights) derived from ellipsoidal heights obtained from GPS observation, by applying Geoid – Ellipsoid separation (N). By using known MSL height of BM, and these height differences, the heights of GPS stations are derived.

Observation Procedure - The procedure to be followed for GPS observations will be depend on the type of GPS receiver being used and the accuracy required. Mainly two types of GPS receiver have been used in Survey of India. Single frequency receivers are used to measure baselines not more than 10 Km for Topographical Surveys generally.. For longer baselines double frequency receivers are used.

The longer the baselines the longer the observation sessions are required. Obtaining a greater number of satellite during a session increases the strength of baseline. A session requiring 1 hour with 4 satellites might require only 45 minutes with 6 satellites. Experiences will be the best judge of the observation session length. However, following table may be considered for the planning of baseline measurements.

Sl. No.	Baseline Length	No. of Satellites available	Time of Observation
		6	45 Minutes
		4	60 Minutes
1.	1 to 10 Kms	3	2 Hours
		6	50 Minutes
		4	90 Minutes
2.	10 to 20 Kms	3	2 Hours 30 Minutes
		6	1 Hours 30 Minutes
		4	2 Hours
3.	More than 20 Kms	3	3 Hours

(Note :- Technical instructions issued by the office from time to time for specific jobs must be followed in place of the table given above.)

For providing Primary Geodetic Control work bases for framework the bases must be observed for 6 hours in case of 25 to 30 Kms but for longer length of bases 12 Hours observation is required.

The recording interval should be 30 seconds.

For special type of surveys like Crustal movement studies and for long base lines of hundreds of kilometers, long observations are required. In G&RB Survey of India for such type of work 24/48/72 hours observations are carried out according to the length of baselines.

A site chart containing a description of the site, sketch of site, weather data and any other information relevant to the observations is prepared at the time of taking the observations.

Data Downloading and Processing - After the receiver have collected the field data, the receivers are connected to the computer using a data downloading cable (RS232) and the data is downloaded into the computer for post processing with the help of downloading software provided with each instrument.

After downloading, the data processing is done with the help of different software supplied with the different receivers e.g.

Ashtech – Processing and downloading software - Prism and GPPS

Leica - Processing and downloading software – SKI

Trimble - Processing and downloading software – GPSurvey and TGO

WM102/101 - Processing and downloading software - Pops

There is scientific software developed by university of Berne named BERNES is the best among the above. BERNES software can process data collected by any receiver, it is more powerful, scientific and gives more precise results. Latest 5.0 version of Bernese software is now available in the market which is windows based. The previous version 4.2 was DOS based.

In BERNES software additional information can be used which can not be used with other software. For example to achieve higher accuracy the following files can be downloaded from internet and can be used during processing.

POLE file or ERP file – contain the **Earth Rotation Parameters.**

PHASIGS file – contain the different receivers and antenna pair parameters.

For processing in BERNES we need data in RINEX format.

What is RINEX format?

RINEX stands for **R**ECEIVER **IN**DEPENDENT **EX**CHANGE

Each receiver type has its own binary data format. As a consequence, data of different receiver type can not easily be processed simultaneously with one particular GPS data processing software package.

To solve this problem, either all manufacturer has to use the same data output format, or a common data format has to be defined that can be used as a data interface between all geodetic receiver type and different processing software system.

Based on development at the University of Berne the Receiver Independent Exchange format was proposed by Grutner at the fifth international geodetic symposium on satellite positioning. RINEX has been accepted by international user community and by the community of the receiver manufacturers. For most of the geodetic receivers the translator software is provided by the manufacturer that converts the receiver dependent data into RINEX format. On the other hand, all major data processing software requires RINEX data as input like BERNES software. RINEX hence serve a general interface between receivers and multipurpose data processing software.

RINEX format consists of 3 ASCII files

1. Observation data file – contain header information and phase and range data.
2. Navigation file - contain ephemeris data
3. Meteorological file - contain Meteorological data

The file names are as follows-

00010731.02o / n / m

0001 stands for point ID No.

073 ,, for Julian day

1 ,, for session No.

02 ,, for year

o ,, for observation / n for navigation etc.

5. STEPS OF COMPUTATIONS TO GET COORDINATES ON ELLIPSOID OF INTEREST -

After processing we get the following results-

- Latitude, Longitude and ellipsoidal height of stations with respect to WGS-84 datum.
- Slope distance or baseline vector between the stations.
- X, Y, Z coordinates with respect to ECEF system.
- Azimuth of the baseline vector.

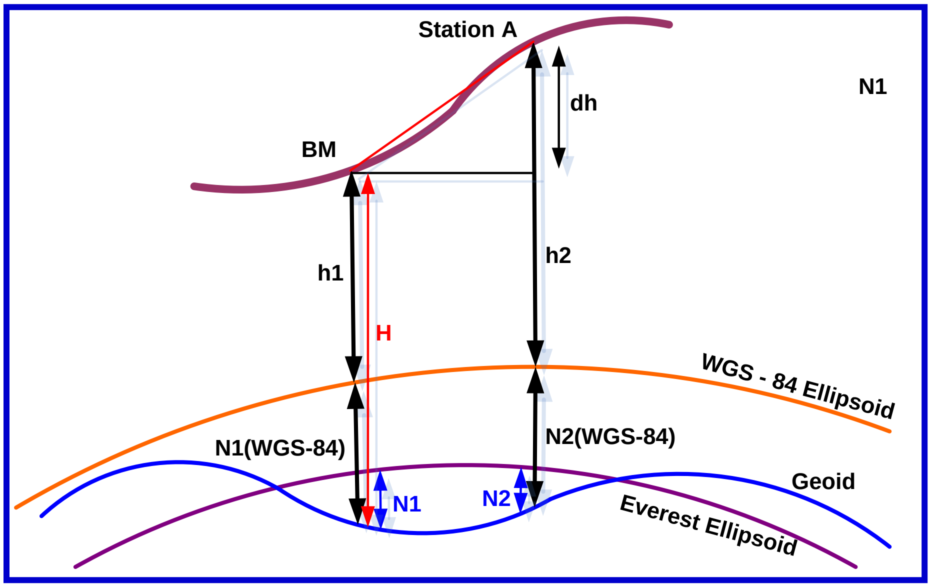
The coordinates obtained after processing are on WGS-84 reference. Survey of India use only baseline vectors. The adjustment carried out to get the co-ordinates on the ellipsoid chosen (Everest etc.). When baseline are used to adjust the network taking one station fixed with known azimuth or two fixed stations. If the vectors are used for the subsequent surveys where GPS observations are not feasible, the baselines observed by GPS must be reduced to the ellipsoidal distances to be used for further traverse or triangulation-

i. Height Adjustment - As we need ellipsoidal height of the stations for reducing slope distances on the Everest ellipsoid, first of all the height adjustment is to be carried out.

Data required for height adjustment:-

- Δh the ellipsoidal height difference of each baseline vector obtained from E-Height (above WGS-84) of stations.
- Baseline length of all measured baselines in km.
- N the Geoidal undulation or separation between Geoid and ellipsoid used (WGS84, Everest etc.).
- Height of at least one Bench Mark above MSL which is connected with GPS network.

How the orthometric heights of stations deduced from known BM height ?



Let the Orthometric of BM = H

Ellipsoidal Height of **BM** above WGS-84 = h_1

Ellipsoidal Height of Station **A** above WGS-84 = h_2

E - Height difference Between BM and Station A= h_2 - h_1 = Δh (Ellipsoidal Height difference)

Geoidal Undulation at BM **=** N_1 **(Between Geoid and Everest Ellipsoid)**

Geoidal Undulation at Station A **=** N_2 **(Between Geoid and Everest Ellipsoid)**

Ellipsoidal Height of BM above Ellipsoid = H - N_1 = h (say)

Ellipsoidal Height of Station A above Ellipsoid = h + Δh = h_A (say)

Orthometric Height of Station A = = h_A + N_2 = H_A (say)

Similarly one can find the orthometric height if geoidal undulation model for WGS-84 is known. The EGM96 is the available geoid model and gives the accuracy of height within 2m.

The program **SOILAP** is used for height adjustment. It can be obtained from Geodetic & Research Branch.

ii. Reduction of Slope Distances on Ellipsoid -

For reducing slope distances on Everest Ellipsoid Program **SLARC** is used. The following input data is required-

- Observed Slope distances
- Height of stations (if orthometric then value of N also)
- Latitude of stations
- Azimuth of each baseline

iii. Adjustment of Network -

To adjust the complete network, the program **SOITAP** is used which is based on variation of coordinate method. The input data required for the program is

- Reduced distances on ellipsoid used
- Provisional coordinates of each station
- Geodetic coordinates of known stations (if only one station is known, then azimuth of a baseline connected to the known station is also required).

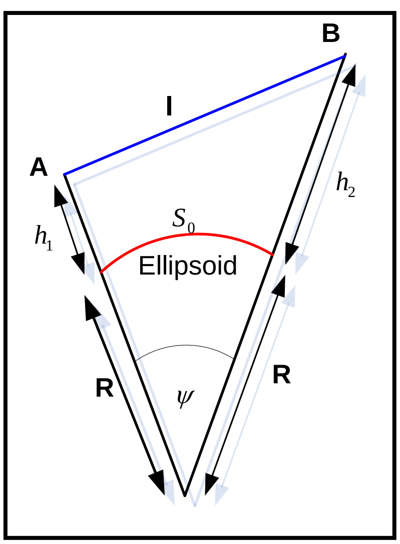
iv. Computation of Latitude and Longitude -

Step 1 :- First of all the heights of all the points adjusted as per triangulation -

- Find dh of all observed vectors. ($dh = h_A - h_B$)
- Make an observation chart and calculate height misclosure of each triangle in the network.
- If there is any discrepancy adjust it as proportional to vector distance.
- Compiled geoidal undulation (N) of each station from the geoidal undulation chart. This chart is available in 19 party (G&RB) and they supply value of N on the basis of Latitude and Longitude of the stations.
- Convert the orthometric height of known station to ellipsoidal height above ellipsoid used by using the formula $h = H + N$.
- On adding the adjusted dh with h we can get the ellipsoidal height of each station.
- The ellipsoidal height is used for reducing the slope distance to arc distance in the next step.
- Find orthometric height of each station using the formula $H = h - N$.

Step 2 :- Reduction of observed slope distances on ellipsoid -

Reductions of a spatial distance on Ellipsoid:-



The following formula can be used to reduced slope distances in to arc distances is –

(Form – 3 Tellumach of SOI)

$$X = R\sqrt{\frac{d^2 - dh^2}{(R+h_A)(R+h_B)}} \qquad = \text{Chord Distance}$$

$Y = \dfrac{X^3}{24R^2}$ = correction to convert chord distance in to arc distance

Arc distance S_0= X + Y

l – Slope distance between station A and B

h_A – Ellipsoidal height of station A

h_B – Ellipsoidal height of station B

dh = $h_A \sim h_B$

R = Mean radius of curvature of Earth for the section AB.

$$R = \frac{\rho\upsilon}{\upsilon\cos^2\alpha + \rho\sin^2\alpha} \ .$$

Where α is the azimuth of the baseline AB

$\rho = a(1-e^2) / (1-e^2\text{Sin}^2\phi_m)^{3/2}$ =Radius of curvature of meridional section

$\upsilon = a / (1-e^2\text{Sin}^2\phi_m)^{1/2}$ = Radius of curvature of Prime Vertical section.

$\phi_m = (\phi_A + \phi_B) / 2$, ϕ_A and ϕ_B can be compiled from map. If map coordinates are not available then the WGS-84 coordinates can be used.

Everest Ellipsoid :

a = 6377301.243 =Semi major axis

b =6356100.231 = Semi minor axis

$e^2 = (a^2 - b^2) / a^2 = 0.00663784607$

WGS84 Ellipsoid :

a = 6378137.0 metre =Semi major axis

b = 6356752.3142 metre = Semi minor axis

$e^2 = (a^2 - b^2) / a^2 = 0.00669438000$

Step3 :- Computation of angles -

The angles of each triangle are computed using the formula $Cos\,A = \dfrac{b^2 + c^2 - a^2}{2bc}$

Where a, b, c are the sides of the triangle.

Step 4 :- Computation of Latitude and Longitude -

To find out the coordinates of unknown stations we must have

- Geodetic coordinates of one station.
- Geodetic azimuth of one base line connected to the known station.

OR

- Geodetic coordinates of two stations.

Survey of India use Clark's formula for precise determination of latitude and longitude. It gives 1ppm accuracy up to 150km. Whereas mid latitude formula for topo. Triangulation gives 8ppm up to 40km. The formula given below is used in Survey of India 13A Trian form.

$$\varphi_B = \varphi_A + \frac{S \times CosA}{\rho_A Sin1''} \qquad \text{(Approximate Latitude of Station B)}$$

$$\rho = \frac{a(1-e^2)}{(1-e^2 \sin^2 \varphi)^{\frac{3}{2}}} \qquad \upsilon = \frac{a}{\sqrt{1-e^2 Sin^2 \varphi}}$$

$$\varepsilon'' = \frac{S^2 \times CosA \times SinA}{2\rho_m \upsilon_m Sin1''}$$

$$\eta'' = \varepsilon'' TanA \times Tan\varphi_B$$

$$d\varphi'' = \frac{S \times Cos(A - \frac{2\varepsilon}{3})}{\rho_m Sin1''} - \eta''$$

$$d\lambda'' = \frac{S \times Sin(A - \frac{\varepsilon}{3}) \times Sec(\varphi_B + \frac{\eta}{3})}{\upsilon_{\varphi_B} Sin1''}$$

$$\Delta A'' = d\lambda'' \times Sin(\varphi_B + \frac{2\eta}{3}) - \varepsilon''$$

$\varphi_B = \varphi_A + d\varphi''$

$\lambda_B = \lambda_A + d\lambda''$

φ_B appearing in $\Delta\lambda''$, ε'' and η'' is the preliminary value of φ_B i.e.

ρ_m in $\Delta\varphi''$ is mean of φ_A and φ_B

φ_B in $\Delta A''$ is the final value of φ_B

Meaning of notations:-

S = distance between station A and B

A = Azimuth at A of B

ρ_m = Radius of curvature of meridian section.

$\nu_{\varphi B}$ = Radius of curvature of P. V. section.

ε = Spherical excess of triangle ABC

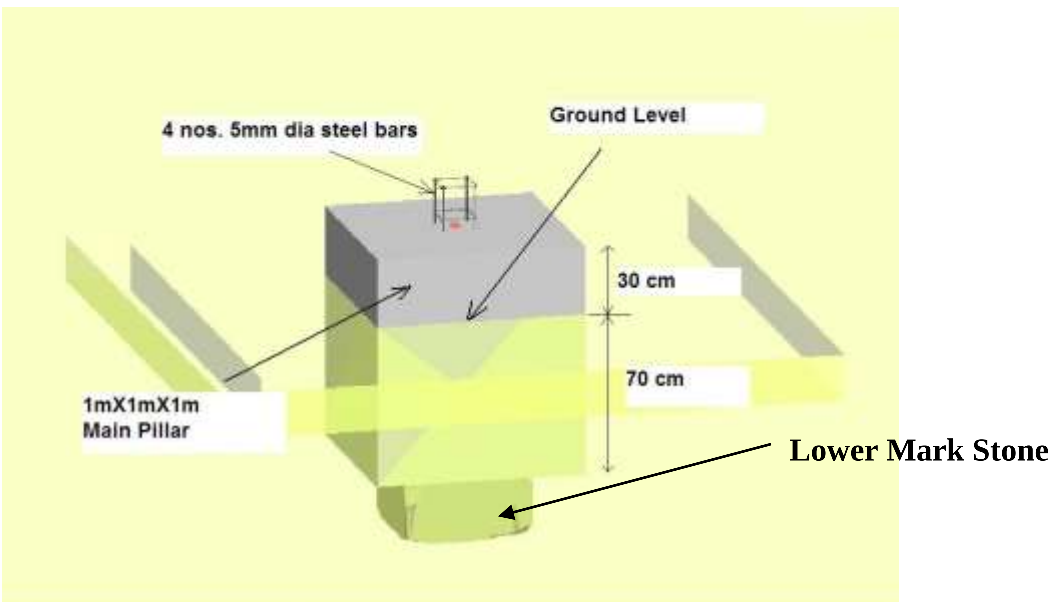
η = Spherical excess of the polar triangle ABC

6. CONSTRUCTION OF GPS PILLAR – MONUMENTATION -

i. Detailed Specifications Of Monument Construction for GCP Library Phase I -

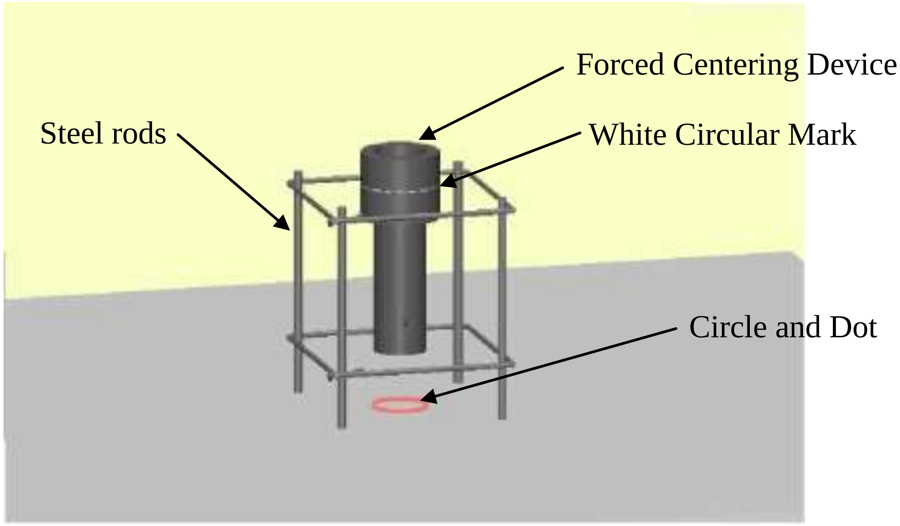
STEP 1.

Embed a large stone engraved with Circle and Dot, in the ground firmly. Construct the main cement concrete pillar of dimension 1m×1m×1m such that it is 0.7m below the ground level and 0.3m above ground. 4 steel bars of 5mm dia and 60 cm length should be embedded in the pillar such that 40 cm of their length is in the concrete. Tie bars of 5mm dia are to be used to tie the four bars. A circle and dot is to be marked on the pillar vertically above the lower mark.



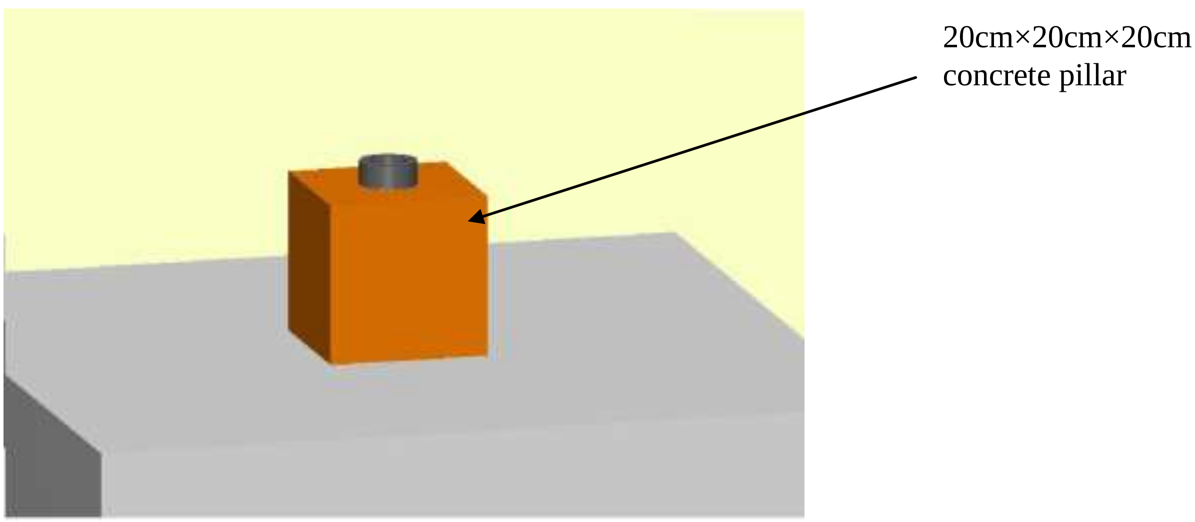
STEP 2.

Place the FCD vertically above the mark such that the white circular mark is at the level where the surface of 20cm×20cm×20cm pillar will come.



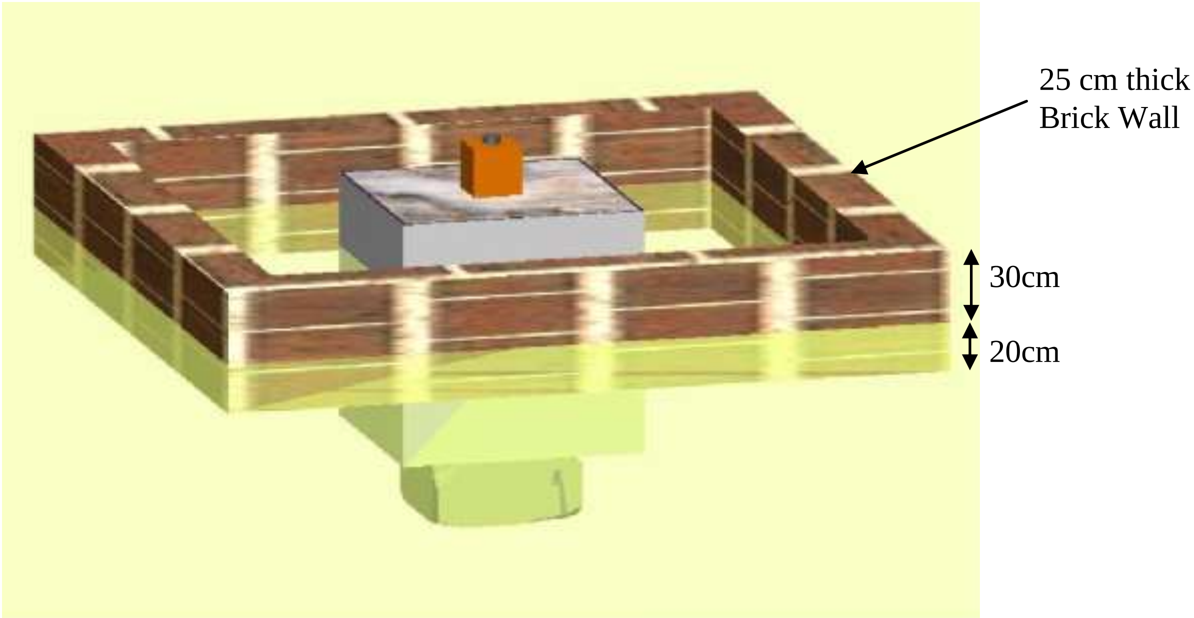
STEP 3.

Construct the cement concrete pillar of 20cm×20cm×20cm such that the surface of the pillar is in the level with white circular mark on Forced Centering Device.



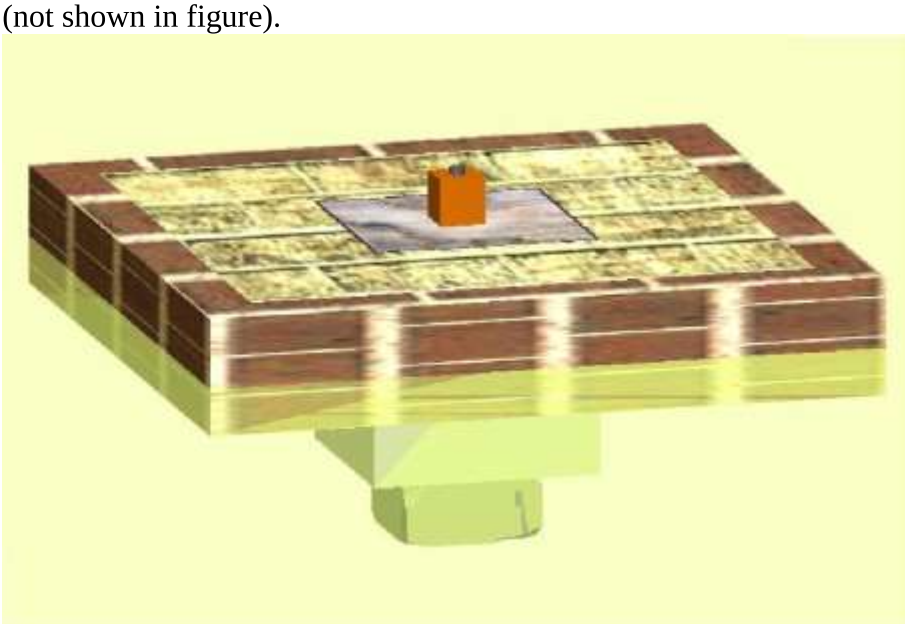
STEP 4.

Construct a brick wall of 25cm thickness to make a square of 3m×3m around the pillar. The wall is to be 50cm high (20 cm under Earth and 30 cm above ground level).

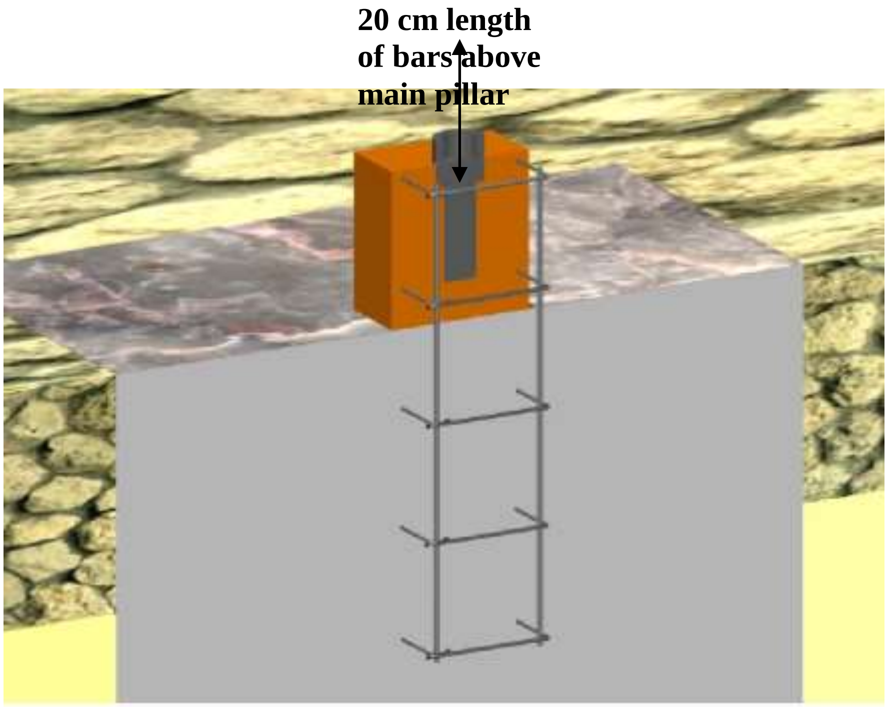


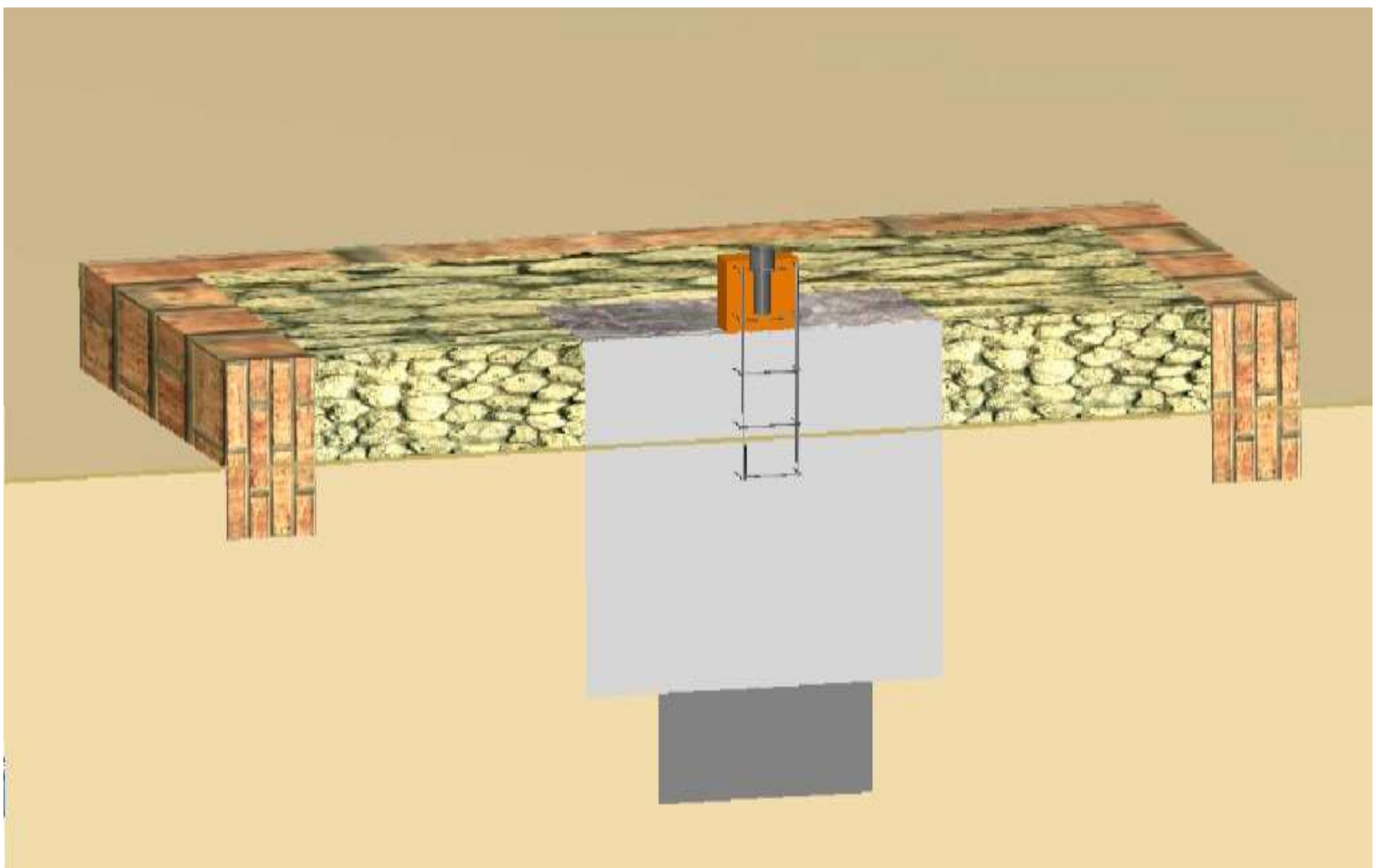
STEP 5.

Construct a platform in between the wall and central pillar, with stones and cement concrete as shown in the figure. Fix marble tiles on the upper surface of central 1m×1m×1m pillar. After construction the platform and the concrete pillar are at the same level and are 30cm above ground. The central 20cm×20cm pillar is 20 cm above the main pillar. Fix angle iron rod or fancy cement pillars of 60cm height on the edges of the platform 60cm apart with fancy chain hanging on pillars (not shown in figure).



The cross sectional view of whole construction is shown below to make it more clear.



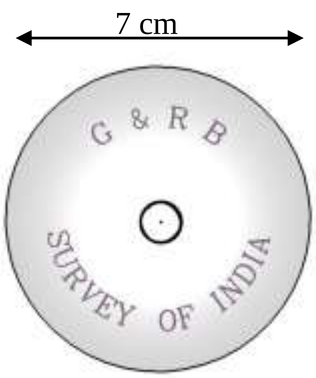


SECTIONAL VIEW OF THE MONUMENT

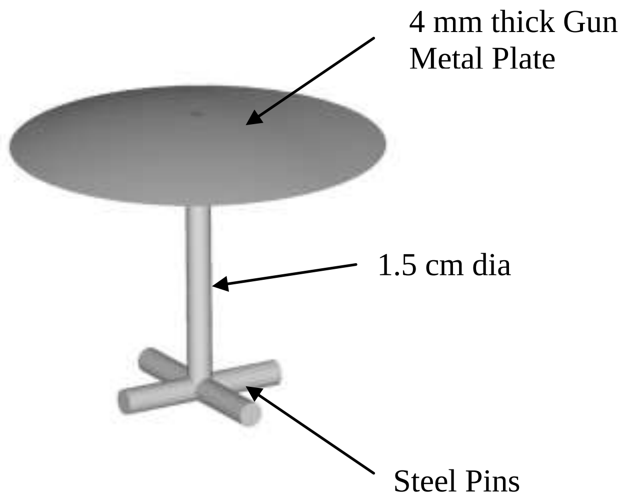
ii. Details of Proposed Pillar for GCP Library - Phase II -

A large stone engraved with Circle and Dot, will be embedded in the ground firmly. It will act as a lower mark. A cement concrete pillar of dimension 1m×1m×1.2m will be constructed in such a manner that it is 0.7m below the ground level and 0.5m above ground.

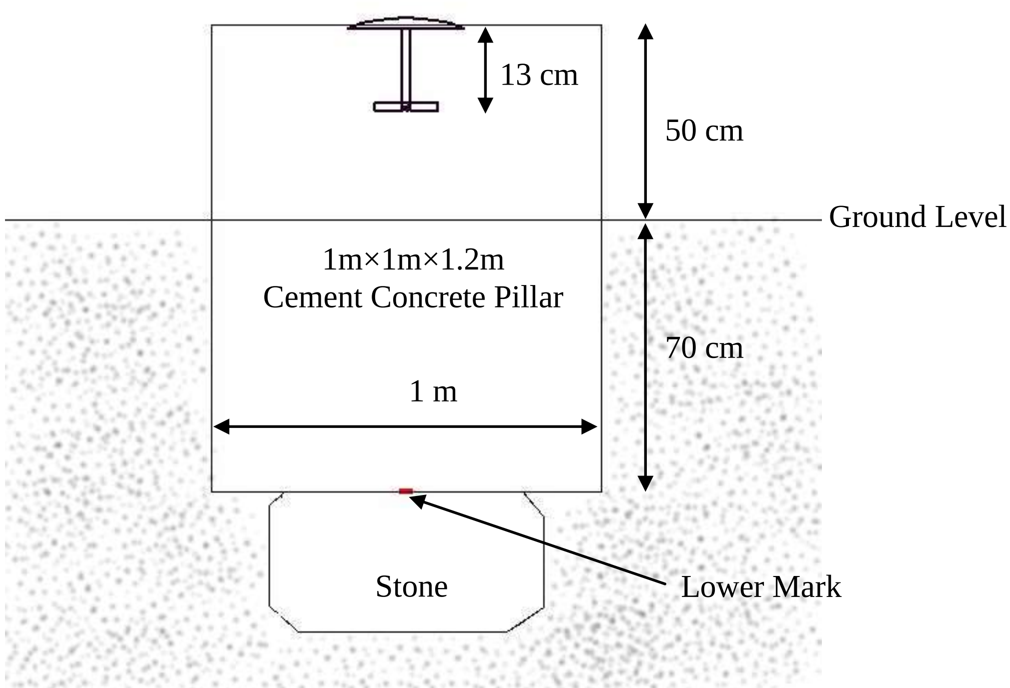
A '**Survey Reference Mark**' made of Gun Metal/brass as shown in the figure will be embedded at the centre of pillar, flushed with the top surface of the pillar.



Top view of Survey Reference Mark

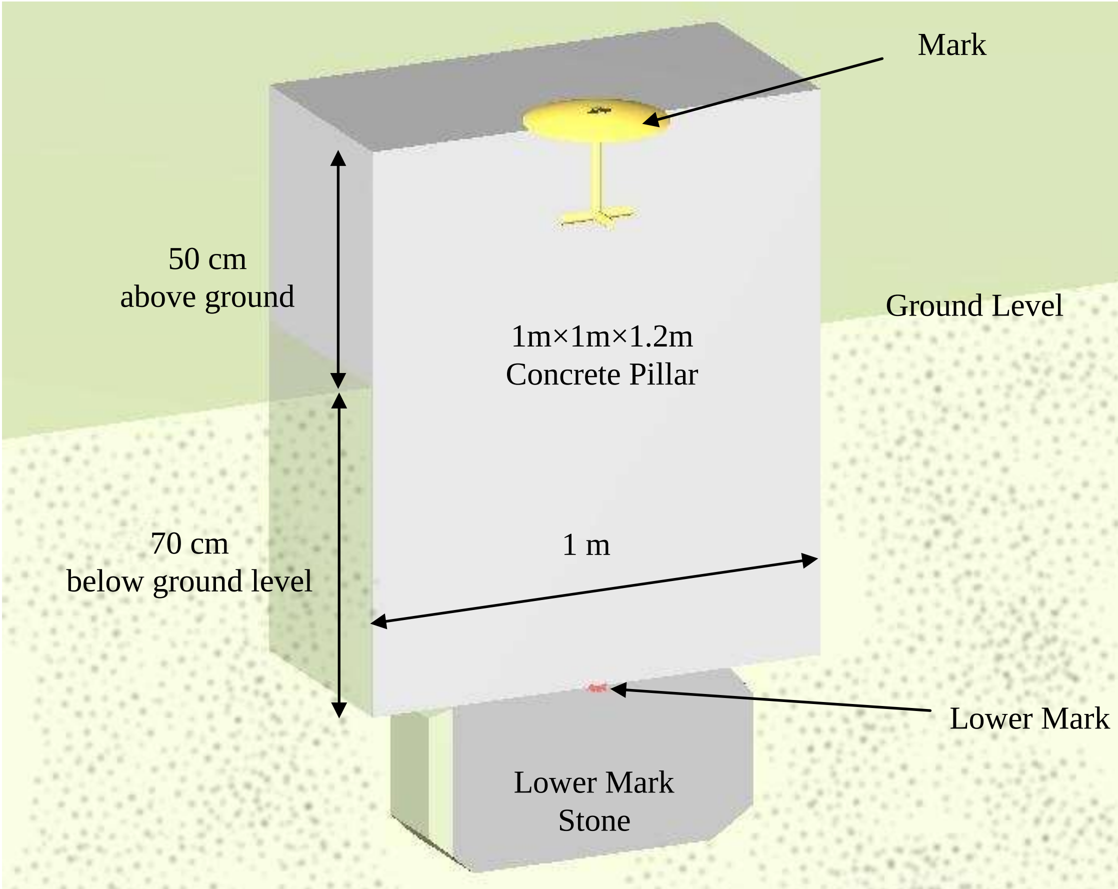


Survey Reference Mark



Sectional View of the Pillar

The cross sectional view given below shows the details.



Cross Sectional View of The Pillar

iii. GPS Stations for Subsequent Surveys -

If the Survey is done for the Projects the pillars of permanent nature must be constructed by the Project Authorities fixing well dressed stone with mark circle and dot at the bottom of the pillar and top of the pillar so that the dot mark of the lower stone must be vertically below the dot mark of the upper stone. The lower stone must be fixed firmly with cement concrete below the Earth and the whole masonry work should be partly below the Earth and partly above the Earth surface.

The galvanized angle iron having a dot at the junction filled with brass can be used in place of stone fixing at the centre of the pillar.

Where rocks with sufficient space to put antenna on tripod is available, the mark consisting of a hole of about 5 mm diameter drilled into the rock about 2 mm deep with a circle engraved around it is marked.

7. EXERCISE FOR FIELD – OBSERVATIONAL PROCESS -

i. Mission Planning -

Planning is a powerful stand-alone software tool supporting any form of analysis of visibility for GPS satellites.

We use the TGO Planning Software before commencing the GPS observation.

- ❖ To define parameters for stations (location, time span and time zone, obstructions)
- ❖ To display visibility & GDOP (Geometrical Dilution of Precision) of satellite graphically.

- ➢ Click on All program - Trimble Office – Utilities – Planning
- ➢ Click on Station or Multistation editor tool and fill up all the information approximately read by map as like Latitude, longitude, Height of the station, Cutoff angle, starting date, time, duration and time zone. Cutoff Angle should be taken as 30° in the valley for above purpose.
- ➢ Click on Station or Multistation – obstacles, to make curtains according to sky obstruction on the station.
- ➢ Download the current Almanac file from receiver.
- ➢ Click on Almanac – Import – Trimble, Select destination path of Almanac File.
- ➢ Click on Graph – No. of Satellites, for satellite visibility.
- ➢ Click on Graph – DOP – Dop Geometrical, for GDOP.

The time of observation will be selected in such a manner that maximum number of satellites is visible and GDOP is less than 5.

ii. Duration of observation -

At the main 4 station which is taken as fixed control for network adjustment will be observed in one session for 24 hours at 15second epoch interval. Other all control points will be observed for 6 to 8 hours at 15 second epoch interval in such a way that every new session will be occupied one common baseline and not in a leap frog method. Height of Antenna will be measured correctly up to millimeter level. Confidence in our surveying results depend upon sufficient redundancy of measurements. Redundancy means number of observables should be more than the number of unknowns, and is essentially required for any proper adjustment process.

Observe as many different independent baselines as possible; it strengthens the adjustment and our confidence too.

We should be preparing a LOG SHEET for each station with following information:

i. Date of observation
ii. Julian Day
iii. Station ID Number
iv. Name of Station
v. Receiver No.
vi. Antenna No.
vii. Starting time
viii. Closing Time
ix. Height of Antenna
x. Description of station with sketch
xi. Name of the Observer with signature.

iii. Downloading the GPS Data from Trimble Receiver -

At the end of day when observation is complete, GPS data must be downloaded in the Laptop with proper management of directory structure etc and a backup copy of the data must be retained before the deletion of the data from the receiver. For downloading the GPS data from receiver to Laptop, we will use the Trimble Data Transfer software as follows:

- Connect the GPS receiver to the Laptop by USB / RS-232 cable and Switch ON the receiver.
- Make a specific folder and subfolder for storing the raw data at the time of downloading. Generally it is created in the name of Julian Day or date of observation.
- Click on the All programs – Trimble Data Transfer – Data Transfer
- Select our device and type of communication i.e. Trimble 5700 GPS receiver on USB port / COM1.
- Click on ADD radio button.
- Click on GPS receiver – file
- Select destination folder with BROWSE option.
- Click on Transfer All
- Selected file will be downloaded in the destination folder.
- File structure of Trimble GPS Receiver 5700 is given below:

  e.g. 95231450.DAT

  Where 9523 is the receiver number
  145 is the Julian day of observation
  0 is the session number
  .DAT is extension for data file.
- For downloading the Almanac file from receiver we should select Almanac file instead of GPS RAW data file.

iv. Conversion of RAW Data into RINEX Format -

The conversion of GPS raw data into RINEX (Receiver INdependent EXchange) format is only required if the data will not be processed by the software given with same make of receiver. In the case of Trimble GPS Receivers, procedure of converting raw data into RINEX format is given below:

- Click on All program – Trimble Office – Utilities – Convert to RINEX.
- Select the Input .Dat file and also select the output folder name by BROWSE option.
- Click on OK
- Select Antenna Type – Zephyr Geodetic
- Select Correction – Correct to Antenna Phase Centre
- Click on EDIT – Measurement Method – Bottom of Notch
- Enter the Original height of Antenna and Click on OK
- Fill up the Header Information
- Enter ID number of station in place of Marker Name
- Click on OK
- Finally Raw Data will convert into RINEX format.

v. GPS Baselines Processing -

The main control points having 24 hours GPS observation, data will be processed with scientific software i.e. BERNESE Software with precise ephemeris. Rest of all baselines will be processed with Trimble Geomatics Office Software.

The Trimble Geomatics Office software provides the ability to generate post-processed GPS baseline solutions and adjustment of network. The following steps are involved:

a. Create a Project.
b. Import the GPS data (Raw / Rinex data / Precise ephemeris) in the project.
c. Baseline Processing
d. Modify the coordinates system manager for Cassini Projection
e. Network Adjustment

a. To create a project:

- Click on All Program – Trimble Office – Trimble Geomatics Office – Trimble Geomatics Office
- Click on New Project – Enter the name of project in Metric Template.
- Select the project folder option.
- Click on OK
- Fill up the Project fields of project properties.
- Click on Units and Format Icons
- By click, make proper change if necessary otherwise default setting gives the result.
- Click on OK

b. Import the GPS data:

i. Trimble Raw Data

- Click on Import – DAT File – Select the destination path of raw data at Look in Box
- Click on Open – Dat Checkin box appear.
- Click on Name box and Change with ID number of station, duplicate number of same station creates trouble, avoid it.
- Enter the height of antenna in Antenna Height Box.
- Select Zephyr Geodetic in Antenna Type Box.
- Select Bottom of Notch to Measured to Box.
- Click on OK
- After all data imported in the project a network figure will be appear on the screen.

ii. RINEX Data

- Click on Import – RINEX File – Select the destination path of RINEX data file at Look In Box.
- Click on Open – Dat Checkin box appear.
- Check all the fields, change if necessary.
- Click on OK
- After all data imported in the project a network figure will be appear on the screen.

iii. Precise Ephemeris:

- Downloading the Precise Ephemeris from Website in SP3 format.
- Address for downloading the precise ephemeris is ftp://igscb.jpl.nasa.gov/pub/product/julianday.
- Click on File – Import – Precise Ephemeris File – OK
- Select the destination path of Precise Ephemeris File in SP3 format in Look in Box
- Select All files and click on Open radio button.
- Precise Ephemeris will be selected for the project for data processing.

c. Baseline Processing:

- Click on Select – All, In network all baselines will be highlighted with Red colour.
- Click on Survey – GPS Processing Styles – Edit
- Change The Elevation Mask says 20°.
- Change Ephemeris – If Precise available otherwise Broadcast is ok.
- Click On OK - OK
- Click on Process Icon or Survey in Tool Bar – Process GPS Baselines, GPS Processing Box will appear.
- Check all processed baselines for the following quality acceptance test criteria
    a) Ratio > 05.0
    b) Reference Variance < 10.0
    c) RMS < 0.010m
    d) Solution Type = Fixed
- Already a sign of √ will appear in the check in box if the above said criteria is achieved.
- Click on Save, A Recompute Report generated automatically.
- Recompute report is examined for the error & warning section, points derivative section, closure section, traverse section and starting points section.
- At the time of analysis of baselines which is not passed, any one of above quality acceptance test criteria that baselines are flagged in red colour.
- Flagged baselines should be examined more closely to determine why they received the flag, and troubleshooting techniques may be required to improve the overall quality of the baseline solution by reprocessing.
- Troubleshooting techniques may or may not improve baselines statistics. Try them one at a time before returning the field.
- Troubleshooting are:
    a) Disabling a satellite
    b) Editing cycle slips
    c) Editing invalid range data
    d) Evaluating and raising the elevation mask.
- Click on red flag baseline
- Select Timeline in tool bar, timeline appears.
- Click on the + sign of the receiver number at the left edge of the timeline window to view the data graphically.
- Identify the data folder containing the data. Use Zoom to Span
- Scan the GPS observation for cycle slips, invalid range errors and other signal loss problems.
- Right-click a satellite icon where the qualitative data is unavailable.
- Select Enable / Disable
- Drag a selection rectangle around the group of GPS observations
- Select Enable / Disable
- The satellite and all associated GPS observations are disabled.
- Reprocess the baseline and compare the processing results with the earlier processing session if the baseline has improved save the baseline solution in the project.
- If the baseline is not improved, view the GPS Baseline Processing report.
- Double-click a baseline solution in the GPS Processing dialog Box or Select a baseline, and click Report.
- Check the Satellite Phase Tracking Summary at each station for breaks or gaps in the L1 or L2 signal. A continuous line indicates a clean signal. A gap or break in the line indicates a cycle slip or satellite signal loss. Tick marks indicate when the receiver picked up the satellite signal after a cycle slip. Remove the data by Timeline method.

- Check the combined Satellite Phase Tracking Summary. The gray shading indicates the measurements actually used in the baseline solution.
- Check the Residual plots for each satellite. These show the RMS (Root Mean Square) for each satellite used to determine the baseline solution. The residual plots indicate the quality of data received from each satellite. You can use this plot to get a feel for the amount of noise in the solution. This plot shows the residuals for each satellite for each measurement cycle. Noisy satellites may affect data coming from other satellites. The lines on the graph should be centered around zero. The amount of noise in the solution is indicated by how far the plot varies from zero. Remove the satellite or disable the GPS observation whose residual plots are far away from the zero.

Reprocess the baseline and analyse the result, if baseline has improved save the baseline in the project.

- If not, re-observe the baseline in the field, no other alternative is left.

Modify The Coordinate System Manager: To define or edit coordinate system, zones, sites, ellipsoid, Geoid models and Cassini projection, use the Coordinate System Manager utility.

- Click on All program – Trimble Office – Utilities – Coordinate System Manager
- Current – Coordinate System Manager Box will appear.
- Click on EDIT – Add Coordinate System Group, Enter the Name (INDIA) and Click on OK
- Click on EDIT – Add Coordinate System – Cassini-Soldner.
- Select Coordinate System Group Name and click on OK, Zone parameter Box appear.
- Enter the name of project, Datum Name WGS-84 click on NEXT, Geoid Model Box Appear
- Select Method – Geoid Grid Model; Model – EGM 96 and click on NEXT, Projection box appear.
- Enter the value of origin for Cassini Projection as central Latitude, Central longitude, False Easting & False Northing and click on NEXT then FINISH.
- One of the GPS stations, which lie in the centre of Network, is taken as origin for Cassini Projection.
- Save and Exit the Coordinate System Manager Utility.
- Changing the coordinate system for a project, open the project – Click on Project
- Click on Project Properties, Project properties Box will appear.
- Click on Coordinate System – Change.
- Select Coordinate System and Zone and click on NEXT.
- Select coordinate system group & Zone and click on NEXT.
- Select Predefine Geoid Model EGM-96 and click on FINISH.

vi. Network Adjustment –

The purpose of performing a least-squares adjustment in network is to estimate and remove random errors, provide a single solution when there is redundant data, minimize corrections made to the observations, detect blunders and large errors, generate information for analysis, including estimates of precision. The two major network adjustment procedures include:

1. Begin with a minimally constrained adjustment (or a free adjustment when inner constraints are used) to adjust and perform a quality-control check of your observations. This step: checks the internal consistency of the network, detects blunders or poor-fitting observations and obtains accurate observation error estimates.

2. Perform a fully constrained adjustment. This step: reference the network to existing control (datum), verifies existing control and obtains accurate coordinate error estimates.

- After reviewing and analyse the all baselines of the network if any flag exits. It is removed by clicking the mouse on the station or baseline, a properties box appears.
- Click on Flag in properties box – Suppress this massage.
- Click on Adjustment – Datum, select WGS-84.
- Click on Adjustment – Adjustment Style, select 95% confidence level.
- Click on OK
- Click on Adjustment – Adjust, for free adjustment of network.
- Re-compute Report is generated automatically.
- Examine the re-compute report for the error & warning section, points derivative section, closure section, traverse section and starting points section.
- Click on Adjustment – Observations – Geoid – Load and OK.
- Click on Adjustment – Points.
- Enter all MSL Heights of the control points in the column of Elevation.
- Click on OK.
- Click on Adjustment – Adjust.
- Click on Adjustment – Points.
- Enter the value of Latitude and Longitude of one or two fixed control points in the proper field and click on OK.
- Click on Adjustment – Adjust.
- If reference factor is high click on observation uncheck the box for Maximum Standard Residual in the form of Outlier.
- Click on Adjustment – Adjust.
- For viewing the network adjustment report click on Reports – Network Adjustment report.
- Click on report – GPS Loop Closure Report, to analyse the loop closure report.
- In network adjustment report generally Chi-square test is failed at 95% confidence level. The Chi-square test is an evaluation of the network reference factor (RF) and the degrees of freedom. If the RF is close to 1.0, the degrees of freedom are acceptable, and the network fits together mathematically, then the network adjustment should pass the Chi-square test.
- The RMS of the coordinates should be less than 0.020 m for a good adjusted solution.

SECTION 2. - TRIANGULATION

1. INTRODUCTION -

With the advent of new Technology, the classical methods of providing control points for survey i.e. Triangulation has been replaced by GPS Surveys, however the areas where GPS Surveys are not feasible due to non-availability of clear sky i.e. in the valleys Triangulation or traverse is carried out based on GPS bases observed. In this chapter an introduction of triangulation is given to carry out the control survey in valley where the breakdown of control survey points are spaced more than 2 Kms apart otherwise EDM / Total station traverse can be done.

Triangulation is the process of measuring the angles of a chain or network of triangles formed by stations marked on the surface of the ground. It is an elementary trigonometric proposition that if the angles of a triangle and the length of one side are known, the lengths of all the other sides can be computed and if direction of one side are known with reference to some system of co- ordinates, it is possible to calculate the co- ordinates of all the other points.

According to its quality, a triangulation is classified as primary, secondary or tertiary. There is also a forth category known as ‘exploratory’ triangulation. In some countries this classification is known as first- order, second- order, third- order and fourth- order.

Primary or First – order triangulation is the highest grade of triangulation and is employed for the determination of the shape and figure of the earth and other geodetic investigation. It also constitutes the basic precise framework for mapping and for less precise triangulation. As it is independent of external checks, all possible precautions and refinements are taken in the observations and their eduction. The length of a base –line in primary triangulation is about 8 to 12km on the average and the side of the triangulation range from 16 to 150km. The average triangulation error (or discrepancy between the sum of the measured angles in a triangle and 180° plus the spherical excess of the triangle) must not be greatly in excess of 1”. The probable error of computed distance should lie between about 1 in 50,000 and 1 in 250,000.

Secondary or second-order triangulation is either one which was planned to be primary, but did not attain the standard of accuracy of that class due to difficulties of observation, or one designed to connect two primary series and thus furnish points closer together than those of primary triangulation. The average triangulation error in secondary triangulation should not normally exceed 3’’. The probable error of distance will vary form 1 in 20,000 to 1 in 50,000.

Tertiary or Third –order (departmentally known a topographical**)** triangulation is run between the stations of the primary and secondary series and forms the immediate control for topographical surveys. The average triangular error may range from 3’’ to 15’’ and the probable error of the compound sides usually lies between 1 in 5,000 to 1 in 20,000.

In the Survey of India, primary and secondary triangulations are termed geodetic and are carried out by units of Geodetic and Research Branch, while tertiary or topographical triangulation is carried out by topographical units.

The heights derived from modern geodetic triangulation, trilateration by GPS or total station are accurate enough for topographical purposes, too much reliance should not be placed on those of the older work, which are not of the same standard of accuracy, owing to the uncertainty of atmospheric refraction.

2. PLANNING FOR TRIANGULATION -

i. Data necessary for commencing a triangulation –

The initial data required for commencing topographical triangulation in an area of which a detail survey is contemplated, is a base of known length, an azimuth, and the latitude, longitude and height above Mean Sea Level of one station or the co-ordinates of the two fixed stations established by GPS. The base length will normally be obtained by GPS or total station (EDM), the azimuth astronomically, and the geographical position and height of one station from previous triangulation or GPS Survey in the vicinity. Scale and azimuth deduced from a GCPs of first phase or second phase may be used if convenient, but not from other existing control points, further connections serve as a check on geographical positions only.

When the survey of a new area is about to be begun, the officer in charge of the party must obtain the GCP pamphlets and charts covering the region, or, if they are not yet published, apply through the Director of his circle, to the Director, Geodetic and Research Branch for the data of any triangulation trilateration ground control point by GPS which might serve as a basis for the topographical framework.

The topographical triangulation is based on this data and is computed in the manner described in the following section.

ii. Topographical Triangulation –

Topographical triangulation should consist of a number of main series of GCP. These main series will generally be about 50 kilometers apart. The main series should be inter- connected by branch or cross-series at about every 25-50 kilometers, to ensure that there will be no accumulation of serious relative error.

Triangulation for extra-departmental work. - In addition to triangulation / trilateration by GPS on which our regular 1: 50,000 or 1: 25,000 scale maps are based, it is often necessary to provide framework control for the production of maps specially required for land development, irrigation and needs of the project for which they are made as cheaply as possible.

Accordingly when planning a triangulation, GPS trilateration framework for extra –departmental mapping, it is first necessary to decide whether :

(a) The scale and accuracy of the resultant maps is such as to make them suitable for incorporation into the regular mapping of the country,

and

(b) Whether the terrain is such as to enable stations to be established and marked with a reasonable chance of their permanent survival.

If the answers to (a) and (b) are in the affirmative then the instruction in this chapter will be rigorously followed. Where, however, the maps are ephemeral in value and the area is so devoid of character as to make the establishment of permanent marks difficult, the methods employed will be such that, while the accuracy essential for the project is maintained, all unnecessary work is cut out. The work will be extended mainly by simple triangles thus ensuring that the maximum progress is achieved with the least work.

In case where the maps are ephemeral in value but the ground is such as to favour the establishment of permanent points, the instruction contained in this chapter will be followed if the time factor permits.

3. PLANNING AND RECONNAISSANCE -

i. Triangulation / Trilateration Programme -

Under the present organization of the Survey of India, each Geo-spatial Data Center (GDC) is allotted a certain area for Survey. This area, in most cases, is of such extent as to employ the party for several years. Having received instructions, from the Director of the GDC as to the order in which the degree sheets covering the area are to be surveyed. The officer-in charge of the party should take such steps as will ensure his topographical triangulation, trilateration, providing GCP programme being kept at least one year in advance of his details survey programme. At the commencement of any field season there should be sufficient points fixed to employ planetablers for the whole season.

When considering his triangulation / trilateration programme, the officer in charge of the party should decide on the general line along which the main topographical series should run, with a view to breaking up the intervals between geodetic series Primary network of GCP in the best way. He should also decide where cross- connections are necessary. If the distance between adjacent geodetic series is more, he should consider where topographical triangulation of ordinary quality is good enough to cross from one series to the other without serious error, or whether he should make a point of carrying triangulation right across the area before plane – tabling is based on it, in order to adjust its closing error first. It may even be necessary to run a special series of better quality across the middle of any exceptionally large area, although this should seldom happen.

ii. Lay-out –

The arrangement of the triangles' of a series must be suited to the nature of the country. A series may consist of simple triangles,or of braced quadrilaterals, or of censed triangles', quadrilaterals and polygons. Simple triangles' generally cover the ground with the least number of stations; the sitting of the stations is simplified by the necessity of seeing only four other stations instead of the five required in a series of braced quadrilaterals; and the long diagonals of the later are avoided. On the other hand, simple triangles do not give the check on the accuracy of the work, which is provided by more complex figures, nor the opportunity of avoiding triangles which turned out to be bad. As a general rule, therefore, chains should consist of braced quadrilaterals, except where circumstances may make a simple triangle or a more complex figure difficult to avoid. An ill – conditioned quadrilateral should not be preferred to pair of well-conditioned triangles, but in such a case the weak diagonal should be observed if possible, in order to provide an independent check.

Ideally, simple triangles should be equilateral, quadrilaterals should be square, and all other figures should be regular. In particular, angle should be small; which in the course of computation particular, will fall opposite a known side, from whichever end of the series the computations may be begun. In simple triangles, such angles should not, so for as possible, be less than 45°. With quadrilaterals this will be impossible, but it should always be possible to compute through a quadrilateral by at least one route involving no angle of less than 30°. In pentagons and hexagons, etc. this limit should be 40°.

It must be noted that opposite a known side is not weak from the point of view of carrying forward the length of the sides, nor does it introduce azimuth weakness unless the short side is so short that reasonable error in centering will cause appreciable angular error, a condition which is unlikely to occur the side is not less than 5 kilometers in length. On the other hand such a triangle is wasteful; it involves the same expenditure of time and accumulation of error as on ordinary triangle, while it makes little progress with the series. A series should only contain triangles of this shape when necessary to surmount particular difficulties.

iii. **Size of triangle and number of points -**

The side of the triangles of topographical triangulation are usually about 8 to 30 km, depending on nature of country and do not normally exceed 30 km in length. In present scenario the triangulation is seldom used for primary control work. The base length may not be 10 Kms in valleys where suitable GPS stations could not be provided.

In order to cover the ground as quickly as possible and shorten the computation, the triangles should be large, provided that a sufficient number of points are obtained for the plane- tabler; this will depend on the scale of survey, the nature of the country, and the state of the atmosphere at different times of the year.

As a general rule, it may be laid down that the number of points, which should be equally distributed within the actual area of a plane-table section, say 40 cm by 40 cm, should not be less than 20 and may occasionally reach a maximum of 40; but as a planet abler requires a number of points falling on the plane-table but outside the area of the section, the triangulation must also fix points outside the area allotted to him. The above holds good whatever the scale of survey may be.

iv. Grazing rays –

The accuracy of triangulation depends entirely on the assumption that the path of ray light contains passing through the atmosphere is curved in the vertical plane, on account of change in the density of the air with height. Lateral refraction (i.e.; curvature in the horizontal plan) will only occur when the air density differs on either side of the ray. Such conditions are clearly unlikely to persist for any length of time at a distance from the ground, but a ray grazing close to the ground (particularly sloping ground) will be very liable to such disturbance.

It is not possible to lay down any permissible tolerance, but grazing rays, clearing the ground by 6 meters or less should be avoided wherever possible. It is important that any such grazing rays, suspected of lateral refraction, should not, when resorted to, from part of a simple triangle: it may then be possible definitely to attribute the closing errors to the weak angle which may be rejected.

v. Reconnaissance –

When a plan for the proposed triangulation has been drawn up and all preliminaries have been settled, it is necessary to reconnoiter the area to fix sites for base - linens and select stations giving the necessary inter-visibility for extension as well as field of view for the fixing of intersected points. The reconnaissance is carried out on a plane table. The sites of stations should be temporarily marked on the ground and a note taken as to the required height of observing pillars and signals, and the direction and amount of clearing required. The triangulator should avoid making stations on tops of houses or close to temples and mosques. It is necessary for the triangulator to visit personally and select every station. If an old map of the area of survey exists, it should be carefully studied before the reconnaissance commences and a preliminary arrangement of the triangulation can be laid out on it. The preliminary arrangement of the triangles laid out on the map must be amended as per the actual ground positions. An all round visibility clearance is essential for the stations chosen so that further extension of control could be possible in any desired direction.

iv. **Construction of Station -**

In order to get the best results from theodolite it is essential to place it on a perfect stable condition. Whenever possible station should be placed on rock-in-situ and so arranged that the legs of the theodolite rests on the rock itself. The mark consists of a hole about 5mm diameter drilled into the rock about 2mm deep with a circle engraved round it.

Where rock is not available, large stone should be embedded about 1m underground with a circle and dot engraved on it, with a second mark vertically above having its dot above the dot of the lower stone, placed flush with the top surface of the platform. It is preferable if both the stones are fixed with the cement and concrete. Surrounding the mark of pillar, a platform should be built of earth and stone at least 0.5m high above ground level and about 3m square keeping survey mark at the centre of the platform.

4. ERRORS IN TRIANGULATION –

Owing to imperfection in observation error develop in very triangulation. As it is essential that every piece of triangulation shall be sufficiently accurate for the purpose for which it is intended, it is very desirable to think out beforehand what magnitude of error, even if exceptional, can be accepted without embarrassment, ad thus decide on the standard and quality of the work that will be necessary.

The accuracy of the series may be judged from the quantities *m* and *p*, where

(i) M is the root-mean-square of an unadjusted horizontal angle (in seconds) as deduced from the triangular errors:

$$M = \sqrt{\frac{\sum \Delta^2}{3n}},$$

$\sum \Delta^2$ being the sum of the squares of all the triangular errors in the series, and n the number of triangles. It should be computed to two decimal places.

All triangles must be included in the computation of m: in a quadrilateral all the four triangles must to include.

It may happen that when computing he series, a triangle or one angle of a triangle has been rejected, on the grounds that it has a large triangular error and is probably unreliable. In such cases the rejected triangle should generally be retained to some cause, which is fairly certainly known not to have affected the other triangles.

(ii) P is the root-mean-square error of the unadjusted difference of height between two stations (in metres), as computed from the closure of height round the three sides of a triangle.

If h1,h2 and h3 are the observed differences of height in the three side of a triangle (each normally 0 being computed from observation at both ends of the ray), it is clear that h1 + h2 +h3 should be zero: let it actually equal ∇.

$$\text{Then } p = \sqrt{\frac{\sum \nabla^2}{3n}},$$

Where n is the number of triangles.

When computing p it is necessary to include only such triangles as are essential for fixing the height of all the stations of the series. It should be computed to two places of decimal.

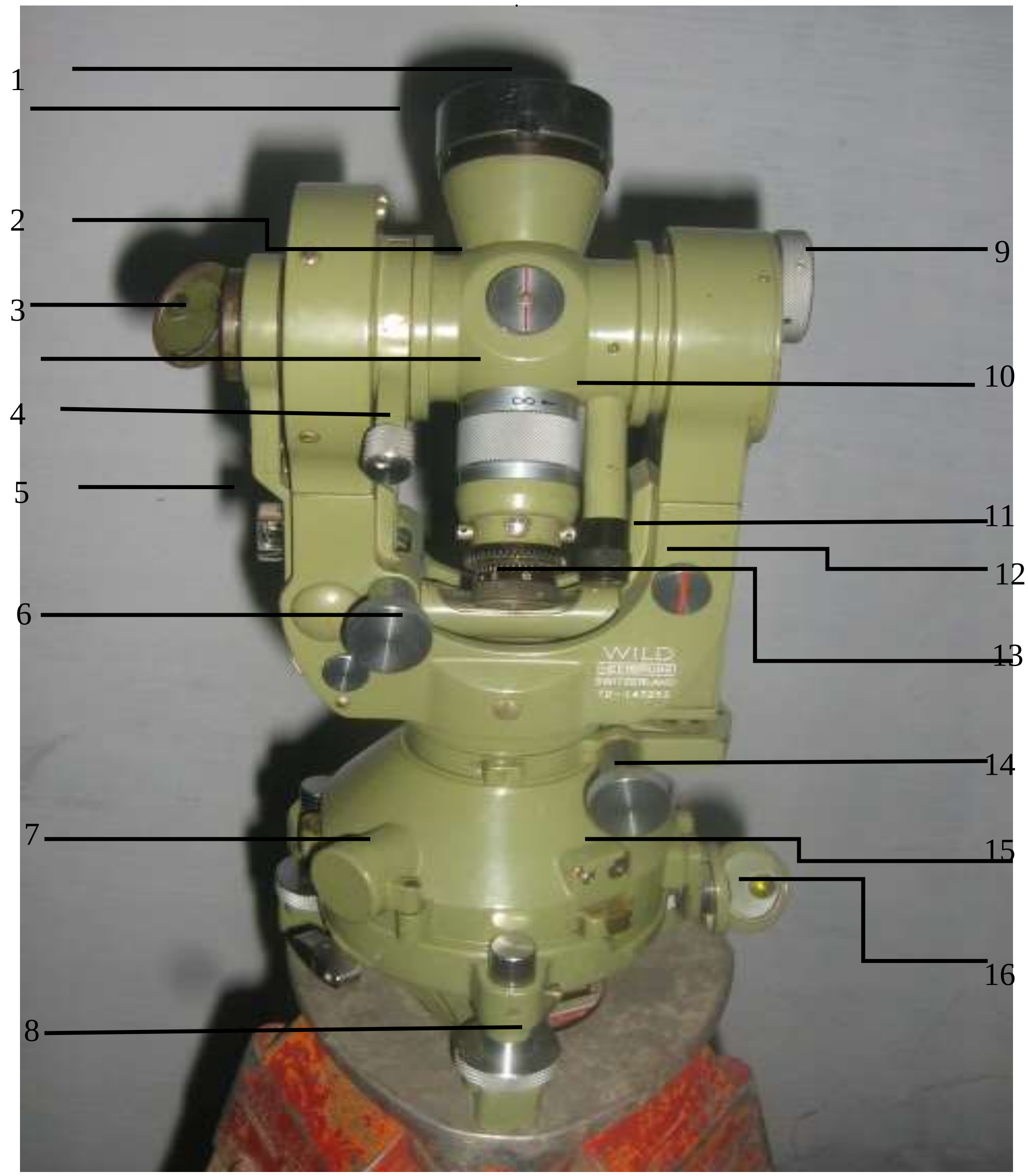
A series with an average triangular error of 8″ will be liable to an error of about 8 meters in latitude or longitude after a distance of 100 kilometers. In 200 kilometers this error is likely to mount up to 17 meters. Stations of topographical triangulation should always be fixed positionally correct within 6 meters. In an ordinary topographical triangulation m would be about half the average triangular error. If a series can be adjusted on to geodetic triangulation at its far end, Its liability to error will be more than halved. If the topographical triangulation is likely to proceed more than 80 kilometers from a geodetic series, the desirability of a adjusting the triangulation before plane-tabling, or of undertaking special triangulation with better instruments, or of inserting measured based of Laplace stations should be considered with a view to fixing the positions to the accuracy required.

The possible error in height must also be considered. It accumulates less rapidly than error of position, but a smaller margin is permissible, especially in flat country. The value of p should be estimated. The probable value of p depends more on the length of the sides and the nature of the country rather than on the quality of the theodolite. Long sides or grazing rays make for larger values of p. Under ordinary circumstances p should not exceed 0.6, but the records of work previously done under similar conditions should be referred to. This ensures that the probable error in height after 100 kilometers will not generally exceed 2 meters. To control the accumulation of large errors in height it is desirable to have at least one station connected by spirit-leveling after every 50 kilometers or so.

5. **INSTRUMENT -**

With the advent of new technology the surveying techniques are drastically changed. The old optical instruments are not in use and so they are rarely available in the market. Survey of India has a good stock of WILD T2 theodolites which were used for triangulation, traverse and were proved more reliable instruments.

The WILD Universal Theodolite



Wild Universal Theodolite

1. Front sight
2. Control for cross-wire illumination
3. Illuminating mirror for vertical circle
4. Vertical clamp
5. Prism for observation of vertical level
6. Vertical tangent screw
7. Circle setting knob (under cap)
8. Foot screw
9. Micrometer Knob
10. Focussing ring of telescope
11. Micrometer eyepiece
12. Change –over knob
13. Plate level
14. Horizontal tangent screw
15. Electric light socket
16. Illuminating mirror for horizontal circle

i. The Wild Universal Theodolite (Model T2) General description -

The telescope of the focusing pattern. Focusing of the telescope is effected by a knurled ring.

The frame carrying he optical and mechanical system for reading the instrument is supported in a conical bearing by the base which also contains the optical system for illuminating the horizontal circle.

The base is supported on three foot-screws which are permanently held in a spring tribrach. The whole instrument is fixed in position by screwing the tribrach on to the head of its stand. An optical plummet and a hook for suspending the plumb-bob are provided for centering the theodolite on the ground mark.

The circles, micrometers and all mechanical and optical parts are completely enclosed and are in fixed adjustment with the exception of the collimation level, the circular base level and the horizontal level, all three of which can be adjusted.

The circles are etched on optical glass and silvered after etching. The horizontal arc is 95mm in diameter and is divided from 0° to 360° by 20′ divisions. The vertical arc is 50mm in diameter, and is divided into 360° by 20′ divisions in the new model and 10′ divisions in the old model. Both the arcs are rigidly connected to the telescope and rotate with it.

There is only one movable horizontal plate. A milled head screw covered by a cap is provided on the tribrach near the base of the theodolite which serves to rotate the plate and thus enables any desired reading to be set in the field of the reading microscope without disturbing the setting of the telescope. On the front and back of the right - hand support are fixed two change over knobs for the horizontal and vertical circles.

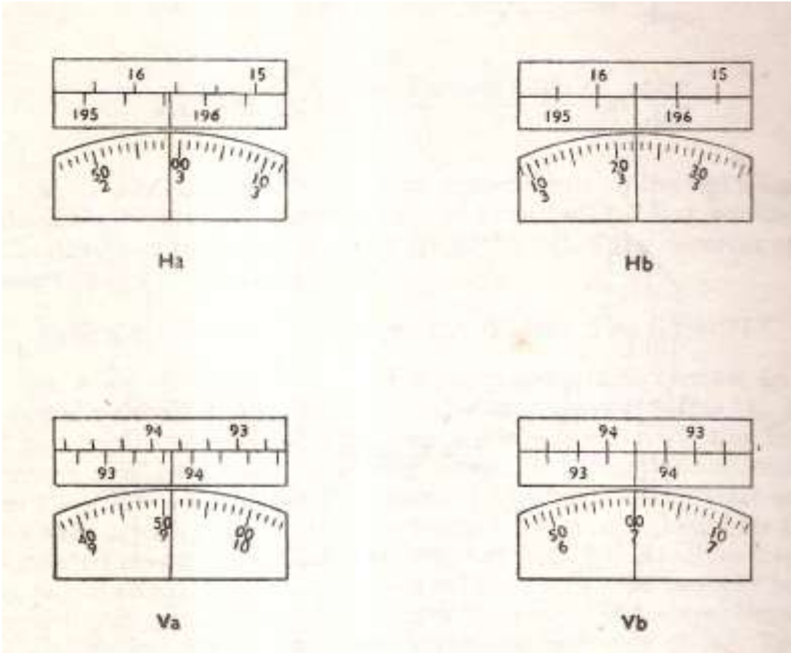
A diagonal eyepiece can be fitted by removing the eyepiece of the telescope and the reading microscope.

For electric illumination, the two illuminating mirrors should be taken out and replaced by the corresponding lamp-holders. If original fittings are available, hang the battery case on the tripod, put the switch in the holder provided for it on one of the tripod legs, and the plug in its socket on the instrument. Otherwise arrange to procure 3 volt dry battery cells, inert-cells or ordinary torch cells arranged in series and parallels to give a uniform 3 volt current and connect the leads of this battery so the plug into the socket of the instrument. Bulbs are required of 2.5 volts strength. For changing the bulbs, the lamp – holders can be drawn out from the illumination supports. A milled head knob on the center of the telescope is provide to regulate the illumination of the telescope diaphragm,

The instrument is packed in a steel case having a base-plate, on which the instrument is mounted firmly with the help of three slides and locking screws, and a steel hood carrying a leather string attached to two locking levers at the base of the hood. The levers are fixed to the rim of the base plate and press the hood firmly to the base plate.

ii. Reading system –

In Wild instruments the two images of the division marks are made to coincide, by and optical device worked by two parallel plate micrometers, which move the divisions by equal amounts in the opposite directions. This motion is indicated by a micrometer drum graduated in minutes and seconds. The circle readings and the micrometer drum readings are seen simultaneously in the reading microscope.



The figure above gives views of a portion of the horizontal circle as seen in the reading microscope immediately after the object is intersected and after an exact coincidence of the graduation lines has been obtained. In the first image the nearest upright figure to the left of the center line gives the degree division 195° and the whole tens of minutes are obtained by counting from this portion to 15° mark, which is diametrically opposite to the division 195°. The fractional part of the division by which the scales have been displaced to bring about the coincidence is given by the reading on the micrometer drum and it is additive to the main scale reading. The reading in the above diagram is 195°43′22.5″.

The reading device of vertical angle is not the same for all models of wild instruments. The instruction book issued with each instrument should be carefully studied to derive the correct values of the vertical angles.

iii. Adjustment of Theodolite –

The adjustment of the theodolite should be performed in the order given below :

a. Centering over a given mark – The centering of small theodolites over a given mark is effected by means or plummet suspended from the centre of the instrument. The plummet should always be centered exactly over the station mark. The adjustment is very easily performed, as the clamping screw of the movable plate of its top can be released, and the theodolite move slightly in any direction, till the plummet is centered exactly over the center of station mark, after which the clamping screw is again tightened.

b. Level Adjustment – An instrument is leveled, i.e, has its vertical axis truly perpendicular to the horizon, when each end of the level bubble remains stable during a complete rotation of the instrument in azimuth. To check it place the level of the horizontal circle of the instrument parallel to a line joining two foot screws and by means of these bring the bubble to the centre of its run, now turn the instrument round exactly 180° in azimuth, then half the error of displacement of the bubble lies in the foot screws, the other half is the error of the level itself and should be corrected by means of the level adjusting screws.

This consists of antagonistic capstan-headed nuts, above and below the level tube, working on a screws thread or merely on a capstan headed screws. The instrument should then again be turned 180° in azimuth into its original position and check the bubble. If the level is still much out of adjustment the process should be repeated.

c. Levelling of the Instrument - Place the level parallel to any two foot screws and bring the bubble to the centre of its run by turning both foot screws simultaneously in opposite direction. Then turn the instrument 180° in azimuth. If the bubble is out move it by means of the same foot screws into the approximate mean of the two positions. Then turn the instrument 90° in azimuth, and by means of third foot screw move the bubble into the approximate mean position already indicated. The process then must be repeated and the bubble must be kept at the true mean position of its run.

d. Collimation, Transit Axis - The line of "collimation" in a telescope is the line joining the centre of the wires to the optical centre of the object-glass. In a theodolite the telescope rotates on the transit axis, and the line of collimation in a telescope thus mounted , should be at right angles to the transit axis; otherwise it will not describe a great circle when the transit axis rotates. Before commencement of field season the theodolite should be sent to the Geodetic & Research Branch for checking of the proper functioning of the theodolite, since the above defects can be removed properly by instrument mechanic or optical engineer.

e. Making the vertical and horizontal wires respectively vertical and horizontal - Since the collimation adjustment is to be made by means of the capstan screws controlling the diaphragm, it is necessary to examine if the hairs are respectively horizontal and vertical when the instrument is leveled up. To verify the horizontal wire, level the instrument ; then set the telescope on a leveling staff or on some distant well-defined object near the horizon, and move the instrument in azimuth, when the object should appear to move along the wire from one extremity of the field to the other. Similarly with the vertical wire, move the telescope in altitude so that a distant object or a suspended plumb line may be appear to traverse the vertical wire. If the object does not remain intersected at all parts of the field, the error ought to be rectified by slackening the capstan screws and rotating the diaphragm until the condition is secured. The relative position of the wires with respect to each other having been fixed by the maker, it is only possible to adjust one wire by means of the diaphragm. This adjustment will inevitably reintroduce collimation error.

f. Setting the level of the vertical arc to the zero of altitude, and collimation in altitude - In dealing with the vertical limb, if everything were in perfect adjustment, we should have the following conditions satisfied:

(i) Vertical angles measured on face left would agree with those measured on the face right.

(ii) When the instrument is level (as judged by the constancy of the reading of one end of the bubble when the theodolite is rotated), the upper bubble would be in the centre of its run.

(iii)The intersection of the cross wires would be in the centre of the field of view of the eyepiece.

iv. Adjustment of Telescope –

a. Distinct Vision - The adjustment for distinct vision of wires is of a personal nature, as it varies with the focus of the observer's eye. The eyepiece must be drawn in or out of its cell until distinct vision of the wires is obtained. To perform the adjustment, direct the telescope to the sky, or hold a piece of white paper obliquely a short distance in front of the telescope, so that the wires being projected on a blank field may be viewed by the eye undistracted by other objects. The wires will be truly in the focus of the eye, when they appear sharply defined and all the little specks of dust on them are seen clear and distinct.

b. Elimination of Parallax - Parallax arises when the image formed by the objective is not situated in the plane of the cross-wires. After the wires have been properly focused, direct the telescope on a distant well-defined object, intersecting it with the vertical wire, and then turn the focusing screw, until that object appears in sharp focus. To test the accuracy of the adjustment, intersect the object with the vertical wire, and move the eye slowly to one side; if the object still appears intersected, there is no Parallax.

The adjustment of the object-glass for solar focus when once properly made is the same for all observers; the adjustment of the eyepiece is a personal adjustment, and varies according to individual eyesight.

c. Making the vertical and horizontal wires respectively vertical and horizontal - Since the collimation adjustment is to be made by means of the capstan screws controlling the diaphragm, it is necessary to examine if the hairs are respectively horizontal and vertical when the instrument is leveled up. To verify the horizontal wire, level the instrument; then set the telescope on a leveling staff or on some distant well-defined object near the horizon, and move the instrument in azimuth, when the object should appear to move along the wire from one extremity of the field to the other. Similarly with the vertical wire, move the telescope in altitude so that a distant object or a suspended plumb line may appear to traverse the vertical wire. If the object does not remain intersected at all parts of the field, the error ought to be rectified by slackening the capstan screws and rotating the diaphragm until the condition is secured. The relative position of the wires with respect to each other having been fixed by the maker, it is only possible to adjust one wire by means of the diaphragm. This adjustment will inevitably reintroduce collimation error. For this type of repairs, the instrument should be sent to the CSMW, Geodetic & Research Branch, Dehradun.

d. Setting the level of the vertical arc to the zero of altitude, and collimation in altitude - In dealing with the vertical limb, if everything were in perfect adjustment, we should have the following conditions satisfied:

(a) Vertical angles measured on face left would agree with those measured on the face right.

(b) When the instrument is level (as judged by the constancy of the reading of one end of the bubble when the theodolite is rotated), the upper bubble would be in the centre of its run.

(c)The intersection of the cross wires would be in the centre of the field of view of the eyepiece.

If there is a vertical collimation vertical reading should then be made on a convenient object on both faces, the instrument having been carefully leveled by the body levels on a firm stand. If the results by each face differ by more than 1 minute, that on either face should be brought into approximate accord with the mean of both faces by use of the clip screws, vernier capstan screws, or other means provided. When this has been done, the vertical bubble should be brought to the centre of its run by means of its terminal capstan screws. Check readings should then be taken and the process repeated if necessary.

The capstan screws should not be tightened up so hard as to strip the thread, but sufficiently tight to prevent shake or liability to become loose. They should then not be tampered with by the observer.

In the case of instruments where the vertical circle is packed up separately from the horizontal, the daily adjustment of the upper bubble, if required, is made by the clip screws.

e. Collimation in azimuth – The normal method of collimation in azimuth is as follows:-

The difference of the reading on opposite faces, as obtained by observing an object alternately face left and face right, gives double the error of collimation, so that, the instrument may be set to the mean reading, and the object intersected by means of the diaphragm screws. The process should be repeated till the intersection remains perfect or very nearly so on changing face. The wild has three adjustment screws, one horizontal and two in a slanting position; the latter should be loosened by exactly the same amount when adjusting.

If the distant fixed object, by means of which the collimation has been determined, is situated above or below the horizon, the collimation error thus obtained must be reduced by multiplying it by the cosine of the elevation or depression of the observed object. On the other hand, the amount of the collimation error at the horizon being known, the azimuthal error at any altitude may be obtained by multiplying by the secant of the altitude. This clearly shows that an error of collimation produces its least effect on azimuthal readings when the altitude of the observed object is nothing. It is moreover evident, that the system of observing with the face of the instrument alternately left and right must necessarily eliminate the effect of the error, because it lies in a contrary direction on the two faces.

6. INSTRUCTIONS FOR OBSERVER –

(a) A theodolite should be turned by the standards and not by the telescope. The movement should always be slow and even.

(b) The theodolite should be turned a few revolutions in altitude and azimuth before starting work.

(c) The theodolite must be kept clean, and frequently dusted with a small brush. In particular the pivots and bearing must be kept clean. Lenses should be dusted lightly with the brush.

(d) Lift theodolite by its standards. Do not force foot screws and clamp tangent screws too hard.

(e) If a helio appears so bright as to dazzle the eye, it should be dimmed by holding a muslin screen in front of, but not touching the object-glass. Screens of single, double and triple muslin should be kept for the purpose.

(f) Whenever observations are interrupted, glass arc theodolites should be taken down and returned to the container.

(g) Cooking fires should be well removed from the station.

(h) The observer must be careful not to place his feet on the pillars on which the theodolite stands and also ensure that his movements round the instrument do not affect the level or the stability of the instrument.

(i) Except when the signal appears to be moving from side to side, the observer must not dwell on his intersection. The intersection should be made carefully but rapidly and readings taken soon after.

(j) When horizontal angle are being observed the vertical axis should be lightly clamped.

(k) The telescope should never overshoot the mark. The final intersection with the vertical wire should be made with the slow motion screw in the direction of the swing, the last motion of the screw being against the spring on swing right and in the direction of the spring on swing left.
If the mark is overshot the movement of the telescope should be continued round till it comes up towards the mark again.

(l) The theodolite should not be carried on its stand from place to place.

(m) Do not force the instrument in the box carelessly so that it rattles when transported. The lid should not be pressing unnecessarily on any part.

(n) Care of instruments – Good work can only be obtained from good instruments well maintained. The utmost care must be taken to ensure that instruments are not damaged, or allowed to get out of adjustment.

7. SIGNALS -

Importance of careful centering.— Whatever be the from of the signal erected over a triangulation station, it is essential that it be symmetrical with respect to the vertical line through the center mark, so that the observation of the signal shall be equivalent to an observation of a plumb-line suspended over the mark.

Luminous signals - Signals are of two kinds, opaque and luminous. The latter consists of heliotropes for use by day and of lamps for night work. Lamps are very rarely employed in triangulation for topographical work, and heliotropes should only be used when the nature of the country, haze or distance render the visibility of opaque signals doubtful.

Opaque signals - Among the opaque signals, the following are the most convenient:-

(i) Pole and brush signal - A very useful opaque signal consists of a straight pole about 2.5 metres long with a bunch of bush-wood or long grass tied symmetrically round the top. This is known as the pole and brush signal and is erected vertically over the mark by heaping a pile of stones, about 1.7 metres in height round the foot of the pole.

A pole and brush should always be erected over every station of observation after it is built, and heliotropes should be instructed always to erect it carefully over the mark-stone in cloudy weather, and also before they quit a station. A rough coat of whitewash over the pile of stones will enable it to be seen much more readily against a dark background.

(ii) Basket signal – Another good from of opaque signal is made of thin laths of bamboo worked into the shape of a deep basket. Two of this are joined together mouth to mouth and fastened to a pole running lengthwise through them. The pole is supported by three other poles with forked ends leaning against it, and firmly fixed in the ground, the intervening spaces between the supports being interlaced with split bamboos. This signal, when in good order, is very clearly visible at long distance.

(iii) Stone cairns.—A cairn of stones built up to a height of 3 metes is also a good signal; if the background is dark the cairn may be whitewashed.

(iv) Beacons of red and white or black and white cloth serve as very good signals.
Other types of signals satisfying local conditions may also be tried.

8. OBSERVATIONS -

Definition of terms used in observing.—A "swing**" is** a continuous motion of the telescope round the circle, beginning and ending with the zero station. The swing may be in either direction, i.e. either "right" (clockwise) or "left" (counter- clockwise).

A "measure" is a determination of the number of degrees minutes and seconds contained in an angle. Each angle must be measured more than once, as a single measure is not to be trusted.

A measure is said to be taken on “face Right” or face Left” according as the vertical circle is to the right or left at the moment of observation.

With the Wild Universal Theodolite, a measure is said to be “Micrometer Right” (M.R.) or “Micrometer Left” (M.L.) according to whether the micrometer microscope is to the right or left of the telescope looking from the eye-piece end. In the ensuing paragraph F.L. and F.R. should be read as M.R. and M.L. respectively when required to apply to the Wild Universal Theodolite.

To obtain one horizontal measure of the angle between two objects, two reading of the horizontal circle are necessary, with the telescope pointed first to the one object and then to the other.
A “set” of horizontal observations of any angle consists of two horizontal measures, one being taken on F.L. and the other F.R.

To obtain one vertical measure, only one reading of the vertical circle is necessary, but in order to eliminate instrumental errors two vertical measures are invariably taken, on of F.L. and the other on F.R.; this is called one set of vertical observations.

The “zero of a measure” is the reading of the micrometer on the horizontal circle when the zero station is intersected.

Number of zeros.—In order to eliminate, as for as possible, the effect of graduation errors in the horizontal circle, measures of horizontal angles to the stations of a topographical triangulation’s main or branch series should be taken to at least three zeros, i.e. 0°, 60°, 120°.

Increase of zeros. -- When **a** triangulator is a beginner the Office-in-charge of the party may increase the number of zeros.

Rule for swings.—In taking a set of observations, the theodolite should be swung right on one face and left on the other face. A convenient rule is “face right, swing left (counterclockwise); face left, swing right (clockwise)”.

i. Method of observing horizontal angles –

The angles at the station are thus:- supposing the observer to be at A, and the signal at B,C,D, E, F to be all visible, the instrument is carefully leveled and the upper plate clamped, so the micrometer reads a few minutes over 0°, the lower plate being unclamped. The instrument is then turned round in azimuth until the selected zero station, B for instance, is intersected. Suppose the telescope to be brought up from the left of B, and turned gently, so that B may enter the field of view, the lower plate must then be firmly clamped and the bisection of B completed by using the slow-motion screw of the lower plate. Now unclamp the upper plate and move the telescope of the object to the left, i.e., in the direction of F, then bring it back until B again enters the field of view, taking care that it does not pass the centre wire. It is passed, the telescope must be swung right round the circle in the direction of C, until it is just short of B. Clamp the upper plate gently and make the final intersection by the slow-motion screw of the upper plate, the final intersection being made against the spring of the slow-motion screw. The micrometer readings are now read and the recorder enters the reading in a fair legible hand in the angle book. The observer should then look again into the telescope to see that B remains bisected. If found correct the upper plate is to be gently unclamped and the telescope turned towards C care being taken not to overshoot it. When turning the instrument the hand should grasp the support of the telescope and not the telescope itself. The upper plate is then clamped gently and the final bisection is made by means of the slow –motion screw of the upper plate and the micrometer readings are again read. The procedure is then repeated for D, E and F. The “swing” is then continued and the zero station B is again observed, the direction motion being always from left to right before the instrument is clamped. This completes one measure of each angle. A comparison of the two readings to B station will test the stability of the instrument*.

A complete round of observations is thus obtained at setting F.L. 0°, by a continuous motion or swing from left to right.

Now unclamp the upper plate and change face by turning the telescope through 180°, in a vertical plane and round 180°, in azimuth, so that the face of the vertical circle if previously to the left hand will now be to the right hand. Clamp the upper plate gently and again intersect station B, this time moving the telescope from the direction of C, using the clamp and slow-motion screw of the upper plate only. This is called setting F.R. 180°, the former position being setting F.L.0°, Then proceed to intersect the other stations as before but in the opposite directions, i.e., - towards F, E, etc; so that the motion or swing of the instrument if from right to left taking care, as before, not to overshoot the mark, using the clamp and slow-motion screw of the upper plate to make the intersections.

*If the instrument has been well clamp and has been stable throughout the swing, and if both the intersection of the zero station have been good, the discrepancy should seldom exceeds the smallest interval to which the instrument is graduated. If it seriously exceeds this amount the observer should look to clamping and stability of the instrument, and consider the advisability of cancelling the observations and repeating the swing. It is a good practice to note down the amount of this closing discrepancy in the angle book immediately after each swing.

ii. Method of observing vertical angles. Time of minimum refraction —

Before commencing vertical observations, an observer must make himself thoroughly acquainted with the method in which the vertical circle of his theodolite is graduated.

Vertical angle to all stations should be measured at the time of minimum refraction, i.e; between 1:45 P.M. and 3:45 P.M. It is imperative that vertical observations taken to and from all stations of the topographical triangulation. They should be taken in sets, a set consisting of one observation F.L. and one F.R. and no observation should be considered completed till it has been taken on both faces. For topographical triangulation, not less than two and preferably three sets of observations should be taken. The observer should first take one complete set of observations to each station, and then proceed to take the second set; if there is time, on completion of the latter, he should take a third set. By this means he will have two or three sets of observations to each station at intervals of about half an hour.

Readings of temperature and time must be made at intervals during the progress of vertical angle measurements. This can best be done when beginning each page of the angle book, both on swing right and on swing left. It is then possible to estimate both time and temperature for any observation for the determination of refraction. The thermometer should be whirled in the shade, care being taken that the bulb does not come in contact with anything.

Importance of ‘ground’ as well as ‘top’ heights.—The vertical angle should always, if possible, be observed to the ground level as well as to the top of every intersected point. The top of an intersected point is usually more convenient to the plane tabler for obtaining clinometer-heights, but the ground level is required for the fair maps and for the plane-tabler to regulate his contours.

Recording height of instrument and objects observed.—The height of the instrument and that of the signal above the upper mark, which is usually flush with the station platform, must always be recorded in the angle book. Particular care should be taken when observing vertical angle to intersected points, to state whether the vertical angle has been taken to the top, or to the ground level, or to both.

Horizontal and vertical angle to stations not to be observed simultaneously.—The practice of observing both horizontal and vertical angles simultaneously, by putting the intersection of the wires on the object and reading both sets of scales, should never be permitted when observing to stations. Not only is the intersection liable to be less accurate, but the extra delay in observing the horizontal round, and the extra handling of the instrument entailed, are vary liable to cause a shifting of the instrument, vitiating the values of the horizontal angles.

iii. Horizontal and Vertical Angle Books —

An angle book, usually consisting of 2 quires of form 3 Topo. or 3A Topo; is used for recording the horizontal and vertical angles of topographical triangulation.

The three sets on different zero settings (0°, 60°, 120°) are observed and the vertical angles are observed as per the design of vertical circle. An abstract is made to get the mean of the horizontal angles as well as the vertical angles. A specimen of preparing abstract of the angles is given in the heading Abstract of the results.

It is most essential, in order to avoid errors and loss of time, that the angle books, in which the measurements made with the theodolite are recorded, should be systematically and clearly kept up. All entries must be made in ink on the spot, no erasures should ever be made in the original observations and all corrections must be initialed by the observer.

SURVEY OF INDIA

WING G.D.C.

Angles taken at A h.s. with Wild T-2 Theodolite

		HORIZONTAL ANGLES			VERTICAL ANGLES				
OBJECT	Face and Zero	A	General Mean	Angle ° ' "	Actual reading	Difference	Elev. + or Dep. -	Mean Angle	Remarks
B h.s.	MR	00° 07' 05"	00° 06' 52"		89° 32' 47"	00° 27' 13"	E	00° 27' 06"	Data of Vertical Angles
	ML	180° 06' 40"			270° 26'58"	00° 26' 58"			Time of comt. : 14.10 hrs
C h.s	MR	76° 53' 18"	76° 53' 05"		90° 59' 41"	00° 59' 41"	D	00° 59' 49"	Temp. at comt. = 12° C
	ML	256° 52' 52"		76° 46' 13"	269° 00' 03"	00° 59' 57"			Pressure at comt. = 70.1Cm
D h.s.	MR	127° 14' 16"	127° 14' 02"		93° 16' 40"	03° 16' 40"	D	03° 16' 50"	Time of Closure : 14:28 hrs
	ML	307° 13' 48"		50° 20' 57"	266° 43' 01"	03° 16' 59"			Temp. at Closure = 12° C
E h.s.	MR	198° 34' 13"	198° 33' 58"		86° 45' 24"	03° 14' 36"	E	03° 14' 28"	Pressure at Closure. = 70.1Cm
	ML	18° 33' 43"		71° 19' 56"	273° 14' 21"	03° 14' 21"			Ht. of Instt. = 1.4m
B	MR	00° 07' 05"	00° 6' 54"		89° 32' 47"				Ht. of Stn. = 3.0m
	ML	180° 06' 42"		161° 32' 56"	89° 32' 47"				Ht. of Cairn = 1.5m

Note : All the Zeros are observed and recorded in the same pattern and mean of all zeros is accepted for computation.

Charts for angle books —A small chart showing the area triangulated should be pasted inside the cover of the angle book.

Entries in angle books —The name of the station of observation should be entered in neat block letters. Names of stations, to which rays are to be observed, should be similarly written in full for the first round. The initial letters of the stations may be used in the second and subsequent rounds, but the full names should be written up at the end of the day's work. A clear space of one or two lines should be left after each round angle; also one blank page at least should be left for the abstract and the description of the station.

Description of stations —The stations of observation must be very carefully described, so that anyone, who wishes to use them later, may have no difficulty in finding and identifying the marks. The description of the mark-stones should come first, then that of the pillar and platform with their dimensions, then a description of any auxiliary marks or satellite stations with their distances and bearings from the station; after these should be entered the directions distances of the neighboring villages and the local name of the spot, if any, also the best means of getting to the station, position of suitable camping ground, water supply available, and the name of the village and district in whose land it is situated, the distance and bearing of the mark tree, and any other information likely to prove useful in the future. This description must be written in the angle book at the end observations at that station, in clear legible writing and all names of place should be hand printed.

When any station is a "pakka" one, or is not of a fairly permanent character, the fact should be noted.

Spelling of names — It is very important that names of all the stations and intersected points, and also of village, hills, etc; entered in the description of stations and intersected points, should be correctly spelt in the angle books, and that they should be the same as on the published sheets.
Abstract of results — The method of preparing the abstract of the horizontal angles is illustrated by an example.

It will be seen, on reference to this, which is an extract from an angle book of all the observation taken at a station that the measures on L. 0 & R. 180 of the two angles between ziarat h.s.- Topkhana h.s. is and Topkhana h.s. Ghari h.s. differ by 13″ and 14″ from the mean of the measures on the other two zeroes; the measures of the other angles being in satisfactory accordance. It is obvious that one or both of the observations to Topkhana h.s. on that zero was bad.

Taking out the measures of the angle between ziarat h.s. and Topkhana h.s. on the separate faces we have:-

				°	'	"
F.L.	0	..	..	50	20	35
F.R.	180			50	20	55
F.L.	60			50	20	55
F.R.	240			50	20	60
F.L.	120			50	20	60
F.R.	300			50	20	60

This shows that the faulty measures was on FL 0. A repeat sets of observations was therefore made on both faces of this zero of both the angles affected and these, being in satisfactory accordance with the measure on the other zero were accepted and the bad measure on FL 0 was rejected.

9. COMPUTATIONAL METHODS -

i. Adjustment of Triangular Error -

The three angles of triangle observed when added must give 180° but in practical, the sum of the three angles differ by few seconds. This error is distributed among all the three angles according to the magnitude of the angles observed. If one angle is nearly 90° the error is distributed in other two acute angles. If the angle is more than 90° the error will be distributed according to the magnitude of the angle excess to the 90°. The maximum triangular error accepted is 10″.

ii. Computation of sides of Triangles -

The sides of the triangle when one side and other three angles of triangle are available can be computed by using simple Sine formulae of trigonometry.

$$\frac{\text{Sin A}}{a} = \frac{\text{Sin B}}{b} = \frac{\text{Sin C}}{c}$$

A,B,C are angles of the triangles and a,b,c are the sides of the triangle.

iii. Computation of the known side of the Triangle –

If two fixed stations are known the distance and bearing between the two stations can be computed by using formulae to compute distance and azimuth of station no. 1 and 2 (Robbin's Reverse Formulae)

1. ϕ_1 Latitude of station no. 1
2. λ_1 Longitude of station no. 1
3. ϕ_2 Latitude of station no. 2
4. λ_2 Longitude of station no. 2
5. $\Delta\lambda = \lambda_2 - \lambda_1$
6. a Semi major axis of ellipsoid (6378137.0 metre (GRS 80))
7. b Semi minor axis of ellipsoid (6356752.3142 metre (GRS 80))
8. e_1 First eccentricity of ellipsoid (0.081819190842622)
9. $\varepsilon = \dfrac{e_1^{\,2}}{1-e_1^{\,2}}$
10. N Radius of curvature in Prime Vertical = $\dfrac{a}{\sqrt{1-e_1^{\,2}Sin^2\phi}}$
11. A_{12} Azimuth at station no. 1 of station no. 2
12. A_{21} Azimuth at station no. 2 of station no. 1 (Azimuth from north clockwise)
13. L Distance between the two points 1 and 2

14. $\Psi_2 = Tan^{-1}\left[(1-e_1^{\,2})Tan\phi_2 + e_1^{\,2}\,\frac{N_1 Sin\phi_1}{N_2 Cos\phi_2}\right]$

15. $\Delta\phi_2 = \phi_2 - \Psi_2$

16. $A_{12} = Tan^{-1}\left[\frac{Sin\Delta\lambda}{Cos\phi_1 Tan\Psi_2 - Sin\phi_1 Cos\Delta\lambda}\right]$

17. $A'_{21} = Tan^{-1}\left[\frac{Sin\Delta\lambda}{Sin\Psi_2 Cos\Delta\lambda - Cos\Psi_2 Tan\phi_1}\right]$

18. $\sigma = Sin^{-1}\left[\frac{Sin\Delta\lambda Cos\Psi_2}{SinA_{12}}\right]$

19. $A_{21} = A'_{21} - \left[\Delta\phi_2 SinA'_{21}\, Tan\left(\frac{\sigma}{2}\right)\right]$

20. $g^2 = \varepsilon Sin^2\phi_1$

21. $h^2 = \varepsilon Cos^2\phi_1 Cos^2 A_{12}$

22. $L = N_1\sigma\left[1 - \frac{\sigma^2 h^2(1-h^2)}{6} + \frac{\sigma^3 gh(1-2h^2)}{8} + \frac{\{\sigma^4(h^2(4-7h^2)-3g^2(1-7h^2))\}}{120} - \frac{\sigma^5 gh}{48}\right]$

(Here σ is in radians)

If A_{12} is near 0° or 180° then line 18 will be calculated as under :

$$\sigma = Sin^{-1}\left[(Cos\phi_1 Tan\Psi_2 - Sin\phi_1 Cos\Delta\lambda)SecA_{12}Cos\Psi_2\right]$$

and $A_{21} = Tan^{-1}\left[\frac{Sin\Delta\lambda}{Cos\phi_2 Tan\Psi_1 - Sin\phi_2 Cos\Delta\lambda}\right]$

Where $Tan\Psi_1 = \left[(1-e_1^{\,2})Tan\phi_1 + e_1^{\,2}\left(\frac{N_2 Sin\phi_2}{N_1 Cos\phi_1}\right)\right]$

N_1 & N_2 are values of N at latitude ϕ_1 & ϕ_2 respectively.

Note : If known baseline is taken from GPS observation, the same should be reduced to the ellipsoid of interest using the formulae illustrated in the GPS Section.

iv. Computation of Co-ordinates of Points -

The azimuth of each line connecting to the known station can be computed by adding or subtracting the angles of observed. The co-ordinates of the points can be computed by using the formulae given in GPS section. However, the simple formulae which is used for Topographical triangulation with small legs is given as follows :

Convergence between the meridians (δA) ϕ
δA = S.P.Q Tanϕ Sin A
Difference of Latitude (δϕ) = S.P. Cos α
Difference of Longitude (δλ) = S.P.Q Secϕ Sinβ
Where S is the length of the side between two stations
A is the azimuth at the fixed station measured from south by west,

P = 1/(Sin 1″) and Q = ρ/ν, Where ρ is the radius of curvature to the meridian at Latitude ϕ and ν is the normal to the meridian at ϕ terminated by the minor axis.

$$\nu = \frac{a}{\sqrt{1 - e_1^{\,2} Sin^2 \phi}}$$

$$\rho = \frac{a(1 - e_1^{\,2})}{(1 - e_1^{\,2} Sin^2 \phi)^{\frac{3}{2}}}$$ (a = semi major axis of Spheroid, e_1 = eccentricity)

α = A+ (1/2) δA
β = A+ δA

δA is positive if A lies between 180° and 360° otherwise negative.

Note : If the observed horizontal angles are available with known side, two fixed stations / one station and observed azimuth, the Survey of India Triangulation Adjustment Programme (SOITAP) can be used to get the adjusted co-ordinates of triangulated points. This programme can be obtained from Director, Geodetic & Research Branch, Survey of India, Dehradun.

v. Computation of Heights -

The computations of heights are done on the basis of the vertical angles observed from station of known height. The reciprocal vertical angles are observed from both the stations. Let A and B be the two stations, the ellipsoidal height of Station A is h_1 and the ellipsoidal distance between station A and B is L.

β is observed vertical angle
R is the radius of curvature at Latitude ϕ and then
Δh (Difference of height from A to B = L (1+(h_1/R) Sin (β - Ω + (θ/2)) Sec (β - Ω + θ)
Ω = kθ, k = 0.07
θ = L/R
The mean value of β - Ω may be used for computations.
To get the height difference from A to B, the height of instrument at A is added and the height of signal at B is subtracted from the computed Δh.
It is advisable to connect the Bench Marks with orthometric heights to get the orthometric heights of the other triangulated stations.