UNIT – 1

CONTROL SURVEYING

Working from whole to part - Horizontal and vertical control methods - Triangulation - Signals - Base line - Instruments and accessores - Corrections - Satellite station - Reduction to centre - Trignometric levelling - Single and reciprocal observations - Modern trends – Bench marking

Horizontal control & its methods:

The horizontal control consists of reference marks of known plan position, from which salient points of designed structures may be set out. For large structures primary and secondary control points are used. The primary control points are triangulation stations. The secondary control points are reference to the primary control stations.

Reference Grid

Reference grids are used for accurate setting out of works of large magnitude. The following types of reference grids are used:

1. Survey Grid
2. Site Grid
3. Structural Grid
4. Secondary Grid

Survey grid is one which is drawn on a survey plan, from the original traverse. Original traverse stations form the control points of the grid. The site grid used by the designer is the one with the help of which actual setting out is done. As far as possible the site grid should be actually the survey grid. All the design points are related in terms of site grid coordinates. The structural grid is used when the structural components of the building are large in numbers and are so positioned that these components cannot be set out from the site grid with sufficient accuracy. The structural grid is set out from the site grid points. The secondary grid is established inside the structure, to establish internal details of the building, which are otherwise not visible directly from the structural grid.

Vertical Control & its Methods:

The vertical control consists of establishment of reference marks of known height relative to some special datum. All levels at the site are normally reduced to the near by bench mark, usually known as master bench mark.

The setting of points in the vertical direction is usually done with the help of following rods:

1. Boning rods and travelers
2. Sight Rails
3. Slope rails or batter boards
4. Profile boards

Boning rods:

A boning rod consist of an upright pole having a horizontal board at its top, forming a 'T 'shaped rod. Boning rods are made in set of three, and many consist of three 'T' shaped rods, each of equal size and shape, or two rods identical to each other and a third one consisting of longer rod with a detachable or movable 'T' piece. The third one is called traveling rod or traveler.

Sight Rails:

A sight rail consist of horizontal cross piece nailed to a single upright or pair of uprights driven into the ground. The upper edge of the cross piece is set to a convenient height above the required plane of the structure, and should be above the ground to enable a man to conveniently align his eyes with the upper edge. A stepped sight rail or double sight rail is used in highly undulating or falling ground.

Slope rails or Batter boards:

hese are used for controlling the side slopes in embankment and in cuttings. These consist of two vertical poles with a sloping board nailed near their top. The slope rails define a plane parallel to the proposed slope of the embankment, but at suitable vertical distance above it. Travelers are used to control the slope during filling operation.

Profile boards :

These are similar to sight rails, but are used to define the corners, or sides of a building. A profile board is erected near each corner peg. Each unit of profile board consists of two verticals, one horizontal board and two cross boards. Nails or saw cuts are placed at the top of the profile boards to define the width of foundation and the line of the outside of the wall

An instrument was set up at P and the angle of elevation to a vane 4 m above the foot of the staff held at Q was 9° 30′. The horizontal distance between P and Q was known to be 2000 metres. Determine the R.L. of the staff station Q given that the R.L. of the instrument axis was 2650.38.

Solution:

Height of vane above the instrument axis

$$= D \tan \alpha = 2000 \tan 9° 30'$$

$$= 334.68 \text{ m}$$

Correction for curvature and refraction

$$C = 0.06735 D^2 \text{ m}, \quad \text{when D is in km}$$

$$= 0.2694 \approx 0.27 \text{ m } (+\text{ve})$$

Height of vane above the instrument axis

$$= 334.68 + 0.27 = 334.95$$

R.L. fo vane = 334.95 + 2650.38 = 2985.33 m

R.L. of Q = 2985.33 – 4 = **2981.33 m**

An instrument was set up at P and the angle of depression to a vane 2 m above the foot of the staff held at Q was 5° 36′. The horizontal distance between P and Q was known to be 3000 metres. Determine the R.L. of the staff station Q given that staff reading on a B.M. of elevation 436.050 was 2.865 metres.

Solution:

The difference in elevation between the vane and the instrument axis

$= D \tan \alpha$

$= 3000 \tan 5° 36' = 294.153$

Combined correction due to cuvature and refraction

$C = 0.06735\ D^2$ metres , when D is in km

$= 0.606$ m.

Since the observed angle is negative, the combined correction due to curvature and refraction is subtractive.

Difference in elevation between the vane and the instrument axis

$= 294.153 – 0.606 = 293.547 = h.$

R.L. of instrument axis = 436.050 + 2.865 = 438.915

∴ R.L. of the vane = R.L. of instrument aixs – h

= 438.915 – 293.547 = 145.368

R.L. of Q = 145.368 – 2

= 143.368 m.

In order to ascertain the elevation of the top (Q) of the signal on a hill, observations were made from two instrument stations P and R at a horizontal distance 100 metres apart, the station P and R being in the line with Q. The angles of elevation of Q at P and R were 28° 42′ and 18° 6′ respectively. The staff reading upon the bench mark of elevation 287.28 were respectively 2.870 and 3.750 when the instrument was at P and at R, the telescope being horizontal. Determine the elevation of the foot of the signal if the height of the signal above its base is 3 metres.

Solution:

Elevation of instrument axis at P = R.L. of B.M. + Staff reading

= 287.28 + 2.870 = 290.15 m

Elevation of instrument axis at R = R.L. of B.M. + staff reading

= 287.28 + 3.750 = 291.03 m

Difference in level of the instrument axes at the two stations

S =291.03 – 290.15 = 0.88 m.

α -- = 28° 42 and α ---- = 18° 6′

s cot α--- = 0.88 cot 18° 6′ = 2.69 m

= 152.1 m.

 h-- = D tan α-- = 152.1 tan 28° 42′ = 83.272 m

 R.L. of foot of signal = R.L. of inst. aixs at P + h-- - ht. of signal

= 290.15 + 83.272 – 3 = **370.422 m.**

Check : (b + D) = 100 + 152.1 m = 252.1 m

h-- = (b + D) tan α-- = 252.1 x tan 18° 6′

= 82.399 m

R.L. of foot of signal = R.L. of inst. axis at R + h--+ ht. of signal

= 291.03 + 82.399 – 3 **= 370.429 m**.

Classification of triangulation system:

The basis of the classification of triangulation figures is the accuracy with which the length and azimuth of a line of the triangulation are determined. Triangulation systems of different accuracies depend on the extent and the purpose of the survey. The accepted grades of triangulation are:

1. First order or Primary Triangulation
2. Second order or Secondary Triangulation
3. Third order or Tertiary Triangulation

First Order or Primary Triangulation:

The first order triangulation is of the highest order and is employed either to determine the earth's figure or to furnish the most precise control points to which secondary triangulation may be connected. The primary triangulation system embraces the vast area (usually the whole of the country). Every precaution is taken in making linear and angular measurements and in performing the reductions. The following are the general specifications of the primary triangulation:

1. Average triangle closure	: Less than 1 second
2. Maximum triangle closure	: Not more than 3 seconds
3. Length of base line	: 5 to 15 kilometers
4. Length of the sides of triangles	: 30 to 150 kilometers
5. Actual error of base	: 1 in 300,000
6. Probable error of base	: 1 in 1,000,000
7. Discrepancy between two measures of a section	: 10 mm kilometers
8. Probable error or computed distance	: 1 in 60,000 to 1 in 250,000
9. Probable error in astronomic azimuth	: 0.5 seconds

Secondary Order or Secondary Triangulation

The secondary triangulation consists of a number of points fixed within the framework of primary triangulation. The stations are fixed at close intervals so that the sizes of the triangles formed are smaller than the primary triangulation. The instruments and methods used are not of the same utmost refinement. The general specifications of the secondary triangulation are:

1. Average triangle closure	: 3 sec
2. Maximum triangle closure	: 8 sec
3. Length of base line	: 1.5 to 5 km
4. Length of sides of triangles	: 8 to 65 km
5. Actual error of base	: 1 in 150,000
6. Probable error of base	: 1 in 500,000
7. Discrepancy between two measures of a section	: 20 mm kilometers
8. Probable error or computed distance	: 1 in 20,000 to 1 in 50,000
9. Probable error in astronomic azimuth	: 2.0 sec

Third Order or Tertiary Triangulation:

The third-order triangulation consists of a number of points fixed within the framework of secondary triangulation, and forms the immediate control for detailed engineering and other surveys. The sizes of the triangles are small and instrument with moderate precision may be used. The specifications for a third-order triangulation are as follows:

1. Average triangle closure	: 6 sec
2. Maximum triangle closure	: 12 sec
3. Length of base line	: 0.5 to 3 km
4. Length of sides of triangles	: 1.5 to 10 km
5. Actual error of base	: 1 in 75, 0000
6. Probable error of base	: 1 in 250,000
7. Discrepancy between two Measures of a section	: 25 mm kilometers
8. Probable error or computed distance	: 1 in 5,000 to 1 in 20,000
9. Probable error in astronomic Azimuth	: 5 sec.

Explain the factors to be considered while selecting base line.

The measurement of base line forms the most important part of the triangulation operations. The base line is laid down with great accuracy of measurement and alignment as it forms the basis for the computations of triangulation system. The length of the base line depends upon the grades of the triangulation. Apart from main base line, several other check bases are also measured at some suitable intervals. In India, ten bases were used, the lengths of the nine bases vary from 6.4 to 7.8 miles and that of the tenth base is 1.7 miles.

Selection of Site for Base Line. Since the accuracy in the measurement of the base line depends upon the site conditions, the following points should be taken into consideration while selecting the site:

1. The site should be fairly level. If, however, the ground is sloping, the slope should be uniform and gentle. Undulating ground should, if possible be avoided.

2. The site should be free from obstructions throughout the whole of the length. The line clearing should be cheap in both labour and compensation.

3. The extremities of the base should be intervisible at ground level.

4. The ground should be reasonably firm and smooth. Water gaps should be few, and if possible not wider than the length of the long wire or tape.

5. The site should suit extension to primary triangulation. This is an important factor since the error in extension is likely to exceed the error in measurement.

In a flat and open country, there is ample choice in the selection of the site and the base may be so selected that it suits the triangulation stations. In rough country, however, the choice is limited and it may sometimes be necessary to select some of the triangulation stations that at suitable for the base line site.

Standards of Length. The ultimate standard to which all modern national standards are referred is the international meter established by the Bureau International der Poids at Measures and kept at the Pavilion de Breteuil, Sevres, with copies allotted to various national surveys. The meter is marked on three platinum-iridium bars kept under standard conditions. One great disadvantage of the standard of length that are made of metal are that they are subject to very small secular change in their dimensions. Accordingly, the meter has now been standardized in terms of wavelength of cadmium light.

UNIT – 2

SURVEY ADJUSTMENTS

Errors - Sources, precautions and corrections - Classification of errors - True and most probable values - weighted observations - Method of equal shifts - Principle of least squares - Normal equation - Correlates - Level nets - Adjustment of simple triangulation networks.

Types of errors.

Errors of measurement are of three kinds: (i) mistakes, (ii) systematic errors, and (iii) accidental errors.

(i) **Mistakes**. Mistakes are errors that arise from inattention, inexperience, carelessness and poor judgment or confusion in the mind of the observer. If a mistake is undetected, it produces a serious effect on the final result. Hence every value to be recorded in the field must be checked by some independent field observation.

(ii) **Systematic Error**. A systematic error is an error that under the same conditions will always be of the same size and sign. A systematic error always follows some definite mathematical or physical law, and a correction can be determined and applied. Such errors are of constant character and are regarded as positive or negative according as they make the result too great or too small. Their effect is therefore, cumulative.

If undetected, systematic errors are very serious. Therefore:

(1) All the surveying equipments must be designed and used so that whenever possible systematic errors will be automatically eliminated and (2) all systematic errors that cannot be surely eliminated by this means must be evaluated and their relationship to the conditions that cause them must be determined. For example, in ordinary levelling, the levelling instrument must first be adjusted so that the line of sight is as nearly horizontal as possible when bubble is centered. Also the horizontal lengths for back sight and foresight from each instrument position should be kept as

nearly equal as possible. In precise levelling, everyday, the actual error of the instrument must be determined by careful peg test, the length of each sight is measured by stadia and a correction to the result is applied.

(iii) Accidental Error. Accidental errors are those which remain after mistakes and systematic errors have been eliminated and are caused by a combination of reasons beyond the ability of the observer to control. They tend sometimes in one direction and some times in the other, i.e., they are equally likely to make the apparent result too large or too small.

An accidental error of a single determination is the difference between (1) the true value of the quantity and (2) a determination that is free from mistakes and systematic errors. Accidental error represents limit of precision in the determination of a value. They obey the laws of chance and therefore, must be handled according to the mathematical laws of probability.

The theory of errors that is discussed in this chapter deals only with the accidental errors after all the known errors are eliminated and accounted for.

The law of accidental errors .

Investigations of observations of various types show that accidental errors follow a definite law, the law of probability. This law defines the occurrence of errors and can be expressed in the form of equation which is used to compute the probable value or the probable precision of a quantity. The most important features of accidental errors which usually occur are:

(i) Small errors tend to be more frequent than the large ones ; that is they are the most probable.

(ii) Positive and negative errors of the same size happen with equal frequency ; that is, they are equally probable.

(iii) Large errors occur infrequently and are impossible.

Principles of least squares.

It is found from the probability equation that the most probable values of a series of errors arising from observations of equal weight are those for which the sum of the squares is a minimum. The fundamental law of least squares is derived from this. According to the principle of least squares, the most probable value of an observed quantity available from a given set of observations is the one for which the sum of the squares of the residual errors is a minimum. When a quantity is being deduced from a series of observations, the residual errors will be the difference between the adopted value and the several observed values,

Let $V1$, $V2$, $V3$ etc. be the observed values

x = most probable value

The laws of weights.

From the method of least squares the following laws of weights are established:

(i) The weight of the arithmetic mean of the measurements of unit weight is equal to the number of observations.

For example, let an angle A be measured six times, the following being the values:

∠ A	Weight	∠ A	Weight
30° 20′ 8”	1	30° 20′ 10”	1
30° 20′ 10”	1	30° 20′ 9”	1
30° 20′ 7”	1	30° 20′ 10”	1

∴ Arithmetic mean

= 30° 20′ + 1/6 (8” + 10” + 7” + 10” + 9” + 10”)

= 30° 20′ 9”.

Weight of arithmetic mean = number of observations = 6.

(2) The weight of the weighted arithmetic mean is equal to the sum of the individual weights.

For example, let an angle A be measured six times, the following being the values :

∠ A	Weight	∠ A	Weight
30° 20′ 8”	2	30° 20′ 10”	3
30° 20′ 10”	3	30° 20′ 9”	4
30° 20′ 6”	2	30° 20′ 10”	2

Sum of weights = 2 + 3 + 2 + 3 + 4 + 2 =16

Arithmetic mean = 30° 20′ + 1/16 (8”X2 + 10” X3+ 7”X2 + 10”X3 + 9” X4+ 10”X2)

= 30° 20′ 9”.

Weight of arithmetic mean = 16.

(3) The weight of algebric sum of two or more quantities is equal to the reciprocals of the individual weights.

For Example angle A = 30° 20′ 8”, Weight 2

B = 15° 20′ 8”, Weight 3

Weight of A + B =

(4) If a quantity of given weight is multiplied by a factor, the weight of the result is obtained by dividing its given weight by the square of the factor.

(5) If a quantity of given weight is divided by a factor, the weight of the result is obtained by multiplying its given weight by the square of the factor.

(6) If a equation is multiplied by its own weight, the weight of the resulting equation is equal to the reciprocal of the weight of the equation.

(7) The weight of the equation remains unchanged, if all the signs of the equation are changed or if the equation is added or subtracted from a constant.

Distribution of error of the field measurement.

Whenever observations are made in the field, it is always necessary to check for the closing error, if any. The closing error should be distributed to the observed quantities. For examples, the sum of the angles measured at a central angle should be 360°, the error should be distributed to the observed angles after giving proper weight age to the observations. The following rules should be applied for the distribution of errors:

(1) The correction to be applied to an observation is inversely proportional to the weight of the observation.

(2) The correction to be applied to an observation is directly proportional to the square of the probable error.

(3) In case of line of levels, the correction to be applied is proportional to the length.

The following are the three angles α, β and y observed at a station P closing the horizon, along with their probable errors of measurement. Determine their corrected values.

α = 78° 12′ 12” ± 2”

β = 136° 48′ 30” ± 4”

y = 144° 59′ 08” ± 5”

Solution.

Sum of the three angles = 359° 59′ 50”

Discrepancy = 10”

Hence each angle is to be increased, and the error of 10” is to be distributed in proportion to the square of the probable error.

Let c1, c2 and c3 be the correction to be applied to the angles α, β and y respectively.

$c1 : c2 : c3 = (2)^2 : (4)^2 : (5)^2 = 4 : 16 : 25$ … (1)

Also, $c1 + c2 + c3 = 10”$ … (2)

From (1), $c2 = 16/4\ c1 = 4c1$

And $c3 = 25/4\ c1$

Substituting these values of c2 and c3 in (2), we get

$c1 + 4c1 + 25/4\ c1 = 10”$

or $c1\ (1 + 4 + 25/4) = 10”$

 $c1 = 10 \times 4/45 = 0”.89$

 $c2 = 4c1 = 3”.36$

And $c3 = 25/4\ c1 = 5”.55$

Check: $c1 + c2 + c3 = 0.”89 + 3”.56 + 5”.55 = 10”$

Hence the corrected angles are

α = 78° 12′ 12” + 0”.89 = 78° 12′ 12”.89

β = 136° 48′ 30” + 3”.56 = 136° 48′ 33”.56

and y = 144° 59′ 08” + 5”.55 = 144° 59′ 13”.55

Sum = 360° 00′ 00”+ 00

An angle A was measured by different persons and the following are the values :

Angle		Number of measurements
65° 30′ 10”	**...**	**2**
65° 29′ 50”	**...**	**3**
65° 30′ 00”	**...**	**3**
65° 30′ 20”	**...**	**4**
65° 30′ 10”	**...**	**3**

Find the most probable value of the angle.

Solution.

As stated earlier, the most probable value of an angle is equal to its weighted arithmetic mean.

65° 30′ 10” x 2 = 131° 00′ 20”

65° 29′ 50” x 3 = 196° 29′ 30”

65° 30′ 00” x 3 = 196° 30′ 00”

65° 30′ 20” x 4 = 262° 01′ 20”

65° 30′ 10” x 3 = 196° 30′ 30”

Sum = 982° 31′ 40”

Σ weight = 2 + 3 + 3 + 4 + 3 = 15

∴ Weighted arithmetic mean

= 982° 31′ 40”

---------------------------- = 65° 30′ 6”.67

15

Hence most probable value of the angle = **65° 30′ 6”.67**

The telescope of a theodilite is fitted with stadia wires. It is required to find the most probable values of the constants C and K of tacheometer. The staff was kept vertical at three points in the field and with of sight horizontal the staff intercepts observed was as follows.

Distance of staff from tacheometer D(m)	Staff intercept S(m)
150	1.495
200	2.000
250	2.505

Solution:

The distance equation is

D = KS + C

The observation equations are

150 = 1.495 K + C

200 = 2.000 K + C

250 = 2.505 K + C

If K and C are the most probable values, then the error of observations are:

150 - 1.495 K - C

200 - 2.000 K - C

250 - 2.505 K – C

By the theory of least squares

$(150 - 1.495\ K - C)^2 + (200 - 2.000\ K - C\ )^2 + (250 - 505\ K - C\ )^2$ = minimum---(i)

For normal equation in K,

Differentiating equation (i) w.r.t. K,

2(-1.495)(150 –1.495 K –C) +2(-20.)(200 - 2.000 K– C)

+2(-2.505)(250 - 505 K – C) = 0

208.41667 – 2.085 K – C = 0 -------- (2)

Normal equation in C

Differentiating equation (i) w.r.t. C,

2(-1.0)(150 –1.495 K –C) +2(-1.0)(200 - 2.000 K– C)

+2(-1.0)(250 - 505 K – C) = 0

200 – 2 K – C = 0 --------- (3)

On solving Equations (2) and (3)

K = 99.0196

C = 1.9608

The distance equation is:

D = 99.0196 S + 1.9608

The following angles were measured at a station O as to close the horizon.

$\angle AOB = 83^o 42'28''.75$ **weight 3**

$\angle BOC = 102^o 15'43''.26$ **weight 2**

$\angle COD = 94^o 38'27''.22$ **weight 4**

$\angle DOA = 79^o 23'23''.77$ **weight 2**

Adjust the angles by method of Correlates.

Solution:

$\angle AOB = 83^o 42'28''.75$ Weight 3

$\angle BOC = 102^o 15'43''.26$ Weight 2

$\angle COD = 94^o 38'27''.22$ Weight 4

$\angle DOA = 79^o 23'23''.77$ Weight 2

Sum = $360^o 00'03.''00$

Hence, the total correction E = 360º - (360º0'3")

= -3"

Let e_1, e_2, e_3 and e_4 be the individual corrections to the four angles respectively. Then by the condition equation, we get

$e_1 + e_2 + e_3 + e_4 = -3"$ -------- (1)

Also, from the least square principle, $\Sigma(we^2)$ = a minimum

$3e_1^2 + 2e_2^2 + 4e_3^2 + 2e_4^2$ = a minimum ------- (2)

Differentiating (1) and (2), we get

$\delta e_1 + \delta e_2 + \delta e_3 + \delta e_4 = 0$ --------- (3)

$$3e_1\delta e_1 + 2e_2\delta e_2 + 4e_3\delta e_3 + 2e_4\delta e_4 = 0 \qquad \text{--------- (4)}$$

Multiplying equation (3) by –λ and adding it to (4), we get

$\delta e_1(3e_1 - \lambda) + \delta e_2(2e_2 - \lambda) + \delta e_3(4e_3 - \lambda) + \delta e_4(2e_4 - \lambda) = 0$ --------- (5)

Since the coefficients of $\delta e_1, \delta e_2, \delta e_3, \delta e_4$ must vanish independently, we have

$3e_1 - \lambda = 0$ or $e_1 = \frac{\lambda}{3}$

$2e_1 - \lambda = 0$ or $e_2 = \frac{\lambda}{2}$ ---------- (6)

`$4e_1 - \lambda = 0$ or $e_3 = \frac{\lambda}{4}$

$2e_1 - \lambda = 0$ or $e_4 = \frac{\lambda}{2}$

Substituting these values in (1), we get

$$\frac{\lambda}{3} + \frac{\lambda}{2} + \frac{\lambda}{4} + \frac{\lambda}{2} = -3$$

$$\lambda\left(\frac{19}{12}\right) = -3$$

$$\lambda = -\frac{3*12}{19}$$

Hence $e_1 = \frac{1}{3} * \frac{3*12}{19} = -\frac{12}{19} = -0.63''$

$$e_2 = \frac{1}{2} * \frac{3*12}{19} = -\frac{18}{19} = -0.95''$$

$$e_3 = \frac{1}{4} * \frac{3*12}{19} = -\frac{9}{19} = -0.47''$$

$$e_4 = \frac{1}{2} * \frac{3*12}{19} = -\frac{18}{19} = -0.95''$$

Sum = -3.0”

Hence the corrected angles

AOB = 83º42’28”.75 – 0”.63 = 83º42’28”.12

BOC = 102º15’43”.26 – 0”.95 = 102º15’42”.31

COD = 94º38’27”.22 – 0”.47 = 94º38’26”.75

DOA = 79º23’23”.77 – 0”.95 = 79º23’22”.82

360º00’00”.00

The following round of angles was observed from central station to the surrounding station of a triangulation survey.

A = 93º43’22” weight 3

B = 74º32’39” weight 2

C = 101º13’44” weight 2

D = 90º29’50” weight 3

In addition, one angle $\overline{(A+B)}$ was measured separately as combined angle with a mean value of 168º16’06” (wt 2).

Determine the most probable values of the angles A, B, C and D.

Solution:

A + B+C+D = 359º59’35”.

Total correction E = 360º - (359º 59’ 35”)

= + 25º

Similarly, $\overline{(A+B)}$ = (A+B)

Hence correction E’ = A + B - $\overline{(A+B)}$

= 168º16’01” – 168º16’06”

= -5”

Let e_1 ,e_2, e_3 ,e_4 and e_5 be the individual corrections to A, B, C, D and $\overline{(A+B)}$ respectively. Then by the condition equation, we get

$e_1 + e_2 + e_3 + e_4 = -25”$ -------- (1(a))

$e_5 - e_1 - e_2 = -5”$ -------- (1(b))

Also, from the least square principle, $\Sigma(we^2)$ = a minimum

$3e_1^2 + 2e_2^2 + 2e_3^2 + 3e_4^2 + 2e_5^2$ = a minimum ------- (2)

Differentiating (1a) (1b) and (2), we get

$\delta e_1 + \delta e_2 + \delta e_3 + \delta e_4 = 0$ --------- (3a)

$\delta e_5 - \delta e_1 - \delta e_2 = 0$ ---------- (3b)

$$3e_1\delta e_1 + 2e_2\delta e_2 + 2e_3\delta e_3 + 3e_4\delta e_4 + 2e_5\delta e_5 = 0 \qquad \text{--------- (4)}$$

Multiplying equation (3a) by $-\lambda_1$, (3b) by $-\lambda_2$ and adding it to (3), we get

$\delta e_1(3e_1-\lambda_1+\lambda_2) + \delta e_2(2e_2 - \lambda_1 + \lambda_2) + \delta e_3(2e_3-\lambda_1) + \delta e_4(3e_4-\lambda_1)$
$+\delta e_5(-\lambda_2 + 2e_5) = 0$ --------- (5)

Since the coefficients of $\delta e_1, \delta e_2, \delta e_3, \delta e_4$ etc. must vanish independently, we have

$$-\lambda_1 + \lambda_2 + 3e_1 = 0 \quad \text{or} \quad e_1 = \frac{\lambda_1}{3} - \frac{\lambda_2}{3}$$

$$-\lambda_1 + \lambda_2 + 2e_2 = 0 \quad \text{or} \quad e_2 = \frac{\lambda_1}{2} - \frac{\lambda_2}{2}$$

$$-\lambda_2 + 2e_3 = 0 \quad \text{or} \quad e_3 = \frac{\lambda_1}{2} \qquad \text{------------ (6)}$$

$$-\lambda_1 + 3e_4 = 0 \quad \text{or} \quad e_4 = \frac{\lambda_1}{3}$$

$$-\lambda_2 + 2e_5 = 0 \quad \text{or} \quad e_5 = -\frac{\lambda_2}{2}$$

Substituting these values of e_1, e_2, e_3, e_4 and e_5 in Equations (1a) and (1b)

$$\frac{\lambda_1}{3} - \frac{\lambda_2}{3} + \frac{\lambda_1}{2} - \frac{\lambda_2}{2} + \frac{\lambda_1}{2} + \frac{\lambda_1}{3} = 25 \quad from(1a)$$

or $$5\frac{\lambda_1}{3} - \frac{5}{6}\lambda_2 = 25$$

$$\frac{\lambda_1}{3} - \frac{1}{6}\lambda_2 = 5 \qquad \text{-------- (I)}$$

$$\frac{\lambda_2}{2} - \frac{\lambda_1}{3} + \frac{\lambda_2}{3} - \frac{\lambda_1}{2} + \frac{\lambda_2}{32} = -5 \quad from(1b)$$

$$4\frac{\lambda_2}{3} - \frac{5}{6}\lambda_1 = -5 \qquad \text{--------- (II)}$$

Solving (I) and (II) simultaneously, we get

$$\lambda_1 = +\frac{210}{11}$$

$$\lambda_2 = +\frac{90}{11}$$

Hence $$e_1 = \frac{1}{3}.\frac{210}{11} - \frac{1}{3}.\frac{90}{11} = +\frac{40''}{11} = +3''.64$$

`

$$e_2 = \frac{1}{2}.\frac{210}{11} - \frac{1}{2}.\frac{90}{11} = +\frac{60''}{11} = +5''.45$$

$$e_3 = \frac{1}{2}.\frac{210}{11} = +\frac{105''}{11} = +9''.55$$

$$e_4 = \frac{1}{3}.\frac{210}{11} = +\frac{70''}{11} = +6''.36$$

Total = +25”.00

Also

$$e_5 = \frac{1}{2}.\frac{90}{11} + 4''09.$$

Hence the corrected angles are

A = 93º43’22” + 3”.64 = 93º43’25”.64

B = 74º32’39” + 5”.45 = 74º32’44”.45

C = 103º13’44” + 9”.55 = 101º13’53”.55

D = 90º29’50” + 6”.36 = 90º29’56”.36

Sum = 360º00’00”.00

1.0 Basic Surveying Principles

Surveying is traditionally defined as the science of making field measurements on or near the surface of the earth to determine relative positions of points. Survey field measurements include horizontal and vertical angles, horizontal and slope distances and vertical distances or height differences.

Generally these points are located by their relationship to each other through a **direction** and **distance**. (For example, Point Y is due East from Point X a distance of 1000 feet.)

A typical survey may require the following:

i. Research

Identifying the location, appropriate survey methods and equipment.

ii. Data Collection

Field measurements and recording of data.

iii. Data Processing

Computations based on the recorded data to determine locations, areas, volumes, slopes and elevations.

iv. Data Representation

Plotting measurements or computed data in map form or reporting such data in a printed format.

v. Stakeout

Setting monuments and stakes to establish boundaries or construction operations.

1.1 Types of Survey Measurements

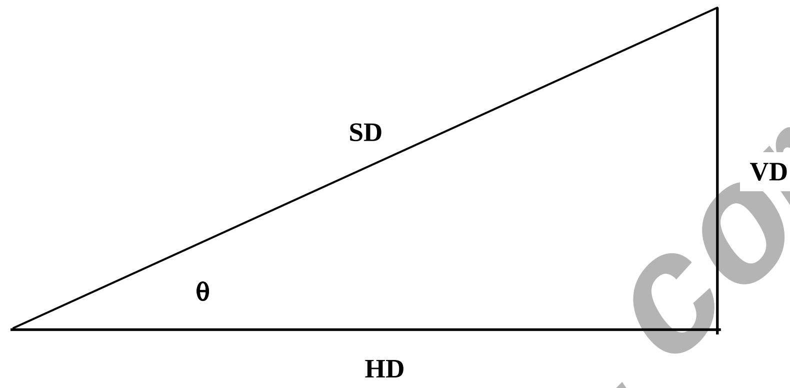

Fig. 1.1: Geometry of Survey Measurements

i. Horizontal Angles HA

Angle measured in the horizontal plane. May be the difference between two directions.

ii. Horizontal Distances HD (=SD cosθ)

Distance computed in the horizontal plane.

iii. Vertical Angles VA (=90° ± θ)

Angle measured in the vertical plane.

iv. Vertical Distances VD (=SD sinθ)

Distances computed in the direction of gravity.

v. Slope Distances SD

Distances measured along inclined planes.

1.2 Units of Measurement

Units	**English**	**Metric**
Length	Feet	Meter
Angle	Degrees, Minutes, Seconds	Degrees, Minutes, Seconds
Area	Square Feet or Acres	Square Meters or Hectares
Volume	Cubic Feet or Cubic Yards	Cubic Meters

Fig. 1.2: Units of Measurement

The location of points and orientation of lines depend on the measurement of angles and directions. In surveying, directions are supplied as bearings and azimuths.

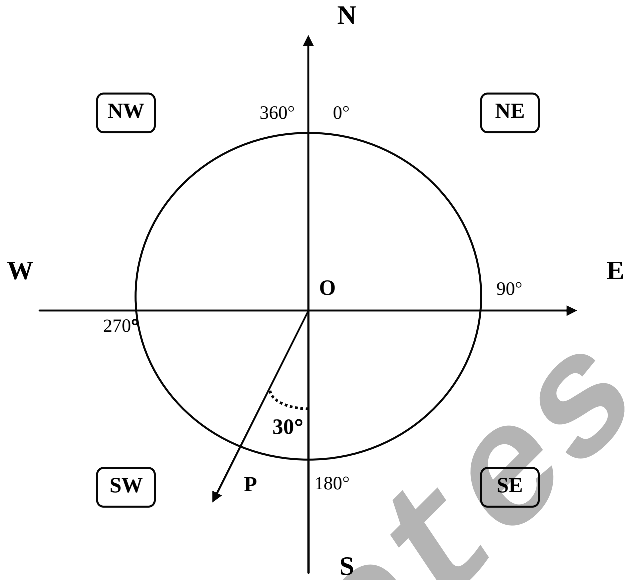

Fig. 1.3: Angles, Azimuths & Bearings

1.3 Angles

Angles are classified as Horizontal or Vertical according to the plane in which they are measured.

Angles are determined by the reference or starting line, the direction of turning and the angular distance or angular value.

Angular values are based upon Degrees, Minutes and Seconds with 360° contained in a circle.

Examples: **$\angle SOP=30°$** **$\angle EOP=120°$**

1.4 Directions

The direction of a line is the horizontal angle between the line and an arbitrarily chosen reference line termed a meridian.

Astronomic (or true) meridian is the North-South reference line through the earth's geographic poles.

Magnetic meridian is the North-South reference line through the earth's magnetic poles.

Assumed meridian is established by assigning any arbitrary direction to a line. Directions of all other lines are found relative to the assumed meridian.

1.5 Bearings

Bearings are a system of expressing directions of lines by means of an angle and quadrant letters. The bearing angle is the acute horizontal angle between the reference meridian and the line.

The angle is measured from either the North or South to the East or West and always gives a reading less than or equal to 90°.

Examples: **OP=S30°W** **ONW=N45°W** **OE=N90°E**

1.6 Azimuths

Azimuths are angles measured clockwise from a reference meridian (generally North) and range from 0° to 360°.

Examples: **OP=210°** **OW=270°** **ON=0° or 360°**

1.7 Distances

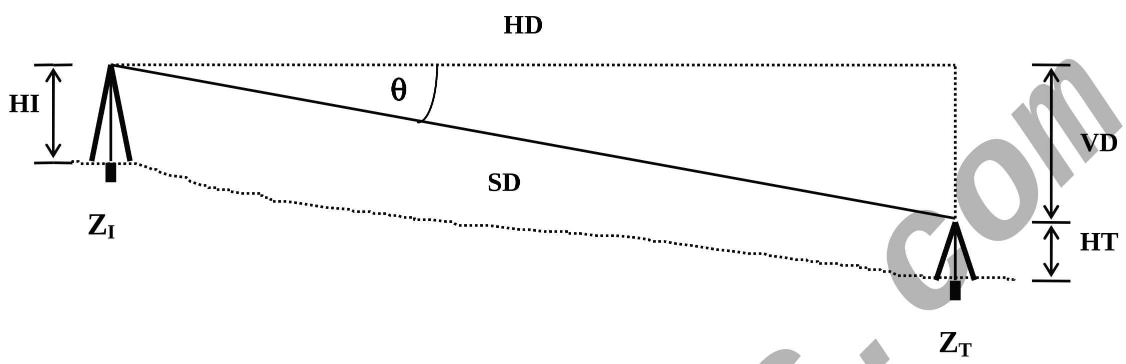

Fig. 1.4: Measurement Geometry

Slope Distance (SD) is the straight line distance from the center of the EDM to the center point of the reflecting prism.

Horizontal Distance (HD) is the straight line distance between plumb lines at the two points.

$$HD = SD \cos\theta$$

Vertical Distance (VD) is the vertical or elevation difference between the center of the EDM and the center point of the reflecting prism.

$$VD = SD \sin\theta$$

1.8 Instrument & Target Heights

The **Instrument Height (HI)** is the height of the instrument above its station.

The **Target Height (HT)** is the height of the prism above its station.

1.9 Elevations

If the elevation of the instrument station **(Z_I)** is known, the instrument and target heights are measured and the slope distance and vertical angles to the target are observed, **elevations** to the target station **(Z_T)** may be computed.

$$Z_T = Z_I + HI - VD - HT$$

1.10 Coordinates and Coordinate Systems

The Rectangular Cartesian Coordinate System is based upon three axes (X, Y and Z) which are perpendicular (at right angles or 90°) to each other.

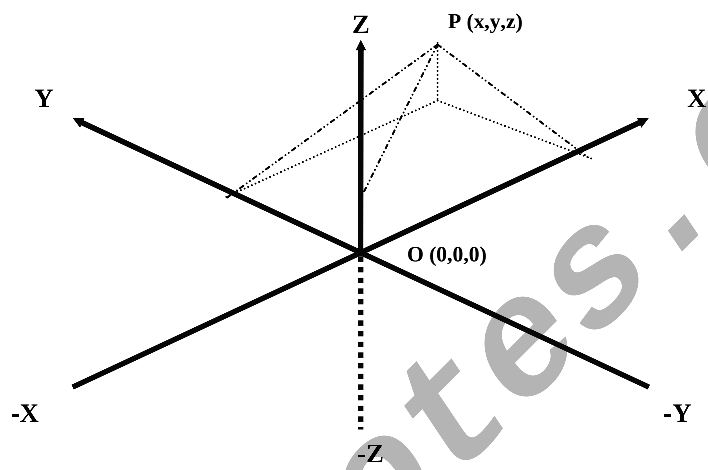

Fig. 1.5: Cartesian Coordinate System Axes

Rectangular Coordinates define the position of a point with respect to the perpendicular axes. The Y-axis points North-South and the X-Axis points East-West and both axes are in the same plane. The Z-axis is perpendicular to the X-Y plane.

1. The Y-Coordinate **(Northing)** is the perpendicular distance from the point to the X-Axis.
2. The X-Coordinate **(Easting)** is the perpendicular distance from the point to the Y-Axis.
3. The Z-Coordinate **(Elevation)** is the distance along the Z-Axis.

Given the rectangular coordinates of a number of points, the relative positions are uniquely defined. Surveying terminology and convention describes the position of a point in terms of **Northing (N), Easting (E) and Elevation (Z)** and is written **(N,E,Z)**.

Examples: **P=(Ny,Ex,Zz) or (y,x,z)**

Coordinates are useful in a number of computations:

i. Calculating lengths and directions of lines.
ii. Calculating areas.
iii. Curve calculations.
iv. Calculating point positions.

Coordinates simplify mapping or plotting tasks.
Coordinates are the most simple and easily handled information in electronic storage devices.

1.11 Types of Surveys

Surveying characteristics depend on the application requirements. Equipment is determined according to survey precision requirements.

Type	Typical Application	Precision Requirements	Percent of Surveys
Geodetic Survey	National surveys Highway Depts. Railways	1:100,000 to 1:1,000,000	10%
Industrial Measurement	Machine Tooling Aircraft	1:100,000 to 1:500,000	5%
Engineering Surveys	Construction Control Photo Control As-built surveys	1:25,000 to 1:100,000	30%
Cadastral Surveys	Subdivisions City Lots Rural Boundaries	1:10,000 to 1:50,000	20%
Mapping	Topographic Cross-sections Profiling	1:5,000 to 1:10,000	30%
Recon Survey	Geology Forestry Route locations	1:50 to 1:5,000	5%

Fig. 1.6: Survey Types & Precision Requirements

Generally require a range of instrument accuracies, though 80% of surveys conducted in the USA (Engineering, Cadastral, Mapping) only require low to medium accuracy instruments.

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2.0 Electronic Total Station Systems

Major instrument manufacturers define a “true” **(Electronic) Total Station** instrument as a fully electronic instrument with coaxial optics which measures horizontal angle, vertical angle and slope distance with a single pointing to the target. Using this measured data, the total station may then compute horizontal distance, vertical distance and two or three dimensional coordinates.

2.1 Total Station Components

A total station is comprised of three fully integrated components;

(i). Electronic Digital Theodolite,
(ii). Electronic Distance Meter (EDM), and
(iii). On-board Microprocessor or computer.

2.2 Electronic Digital Theodolite

The electronic digital theodolite component automatically measures and displays (in digital format) horizontal and vertical angles. Potential human error in reading graduated circles, optical micrometers and vernier scales is eliminated.

2.3 Electronic Distance Meter (EDM)

The electronic distance meter (EDM) is used to measure slope distance from the center of the objective lens of the instrument telescope to the center of the target. The EDM is generally contained in the telescope housing and emits a beam which is coaxial with the telescope optics (or line of sight).

There are two types of EDM technology based upon the signal source;

(i). **Infrared Light Emitting Diode (LED)** uses a continuous signal and measures the phase difference or shift between the transmitted signal from the EDM and the refected signal from the target to compute distance. A reflecting target such as a prism or reflector sheet is used.

(ii). **Pulse Laser Diode** transmits a timed-pulse infrared signal and measures the time required for the pulse of infrared light to travel from the instrument to the target and return to determine the distance. Using this technology, a prism is not necessary, the measured surface is used as the reflecting target.

Pulse laser technology has limited distance capability depending on the power of the laser diode and the reflective properties of the target surface.

2.4 On-board Microprocessor

The microprocessor(s) in an electronic total station can perform a variety of functions and mathematical operations including;

(i). Reduction of vertical angle and slope distance to horizontal and vertical distance.
(ii). Determination of X, Y, Z (N,E,Z) coordinates.
(iii). Coordinate geometry functions - inverses, intersections, etc.
(iv). Remote distance computation.
(v). Remote elevation determination.
(vi). Atmospheric and instrument corrections.
(vii) Recording measured and computed data internally.

2.5 Accessories

2.5.1 Tripod

The tripod is used to securely mount the total station (or a prism assembly) over a point at a height comfortable to the user

2.5.2 Prism Reflector

The reflecting prism is solid glass having a flat front panel and a prism shaped rear. The EDM transmitted signal enters the front of the prism and is reflected back to the instrument by the prism's rear surface. The prism is contained in a housing which is screwed into a targetting frame.

2.5.3 Prism Pole

The prism pole or rod is used to mount the targetting frame of the prism. At the other end of the pole is a sharp, replaceable point which is placed directly on the target, the prism facing the instrument and measurements taken. A circular level vial or bull's-eye bubble attached to the pole is used to ensure the rod is plumb. Most prism poles are telescopic via a quick release clamp to raise or lower the height of the prism (for line of sight issues).

3.0 ACCURACIES IN TOTAL STATIONS

Total stations are specified by angular accuracy and EDM range and accuracy.

3.1 Angular Accuracy

The accuracy or standard deviation of a direction, direct and reversed, (DIN specification) is an indication of how good the theodolite component of a total station is. This should not be confused with the smallest displayable horizontal circle unit which often gives a misleading impression as to the accuracy of the instrument.

A circle has	360° (degrees).		
	1° (degree)	=	60' (minutes).
	1' (minute)	=	60" (seconds).
Thus, a circle has	360x60x60	=	1,296,000" (seconds) of arc.

The **Accuracy** of a total station may be specified as 5". This means, any direction measured will lie within ±5" of the mean of the direct and reverse readings of that direction (at the 68% confidence level). When measuring an angle, two directions are being measured - a backsight and a foresight which results in an angle accuracy of about 7" (√2 x 5").

Angular accuracy of directions over a specified distance may be converted to a distance value as shown in the table below:

Angle Accuracy	**50ft.**	**Distance to Target 100ft. Errors in Feet (Inches)**	**500ft.**
20"	0.0048' **(1/16in)**	0.0096' **(1/8in)**	0.048' **(9/16in)**
10"	0.0024' **(1/32in)**	0.0048' **(1/16in)**	0.024' **(1/4in)**
5"	0.0012' **(1/64in)**	0.0024' **(1/32in)**	0.012' **(1/8in)**
1"	0.0002' **(1/400in)**	0.0004' **(1/200in)**	0.002' **(1/40in)**

Fig. 3.1: Angular Accuracy in Linear Units

(Note: Conversions to fractions of an inch are approximate.)

3.2 Distance (EDM) Accuracy

Electronic Distance Meter (EDM) accuracy is measured in millimeters and parts per million (ppm). For example, ±(5+5ppm)mm or ±(0.02ft+5ppm) where the ±5mm (0.02ft.) is the instrument error which is independent of the length of the measurement and the 5ppm is the distance-related error.

A typical construction grade instrument may have an accuracy specification of ±(5+5ppm)mm. A high order survey instrument may be specified at ±(2+2ppm)mm.

The "± " in the specification indicates the EDM may measure short or long by this amount but the measured distance will fall within this range.

EDM accuracy over a specified distance may be converted to a distance value as shown in the table below:

EDM Accuracy	**50ft.**	**Distance to Target 100ft. Errors in Feet (Inches)**	**500ft.**
±(5+10ppm)mm	0.0169' **(7/32in)**	0.0174' **(7/32in)**	0.0214' **(1/4in)**
±(5+ 5ppm)mm	0.0166' **(7/32in)**	0.0169' **(7/32in)**	0.0189' **(7/32in)**
±(5+ 3ppm)mm	0.0166' **(7/32in)**	0.0167' **(7/32in)**	0.0179' **(7/32in)**
±(3+ 2ppm)mm	0.0100' **(1/8in)**	0.0100' **(1/8in)**	0.0110' **(1/8in)**
±(2+ 2ppm)mm	0.0067' **(3/32in)**	0.0068' **(3/32in)**	0.0076' **(3/32in)**

Fig. 3.2: EDM Accuracy in Linear Units

(Note: Conversions to fractions of an inch are approximate.)

3.3 Combined EDM and Angle Accuracy

The angle and EDM accuracies of a total station may be combined to give the maximum measurement error in a point's location due to the instrument's accuracy specifications.

Instrument	Distance to Target: 50ft.	100ft.	500ft.
	Point Location (Inches)		
20” **±(5+10ppm)mm**	7/16” 1/8”	7/16” 1/4”	7/16” 1”
10” **±(5+5ppm)mm**	7/16” 1/16”	7/16” 1/8”	7/16” 1/2”
5” **±(5+3ppm)mm**	7/16” 1/32”	7/16” 1/16”	7/16” 1/4”
5” **±(3+2ppm)mm**	1/4” 1/32”	1/4” 1/16”	1/4” 1/4”
1” **±(2+2ppm)mm**	3/16” <1/64”	3/16” 1/64”	3/16” 3/64”

Fig. 3.3: Combined EDM and Angular Accuracies
(Note: Conversions to fractions of an inch are approximate.)

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4.0 Instrument Setup

4.1 Setting up the Tripod

- Extend all three legs of the tripod until they are about the same length.
- Set the tripod over the point.
- Open the legs wide enough for the instrument to be stable.
- Ensure the point is directly below the center hole in the tripod head.
- Firmly "plant" one of the tripod ferrules into the ground by stepping on the footplate.
- Ensure the top surface of the tripod head is roughly level.

4.2 Centering over the Point

Centering is the precise alignment of the instrument's central axis over a point.

- Set the instrument on the head of the tripod.
- With one hand holding the instrument, insert the tripod mounting screw into the center hole of the instrument's base plate and tighten.
- Ensure the instrument base plate is centered on the tripod head with the footscrews at the corners of the tripod head.
- Ensure the leveling screws are at the center of their adjustment range.
- Stand between the two legs of the tripod not "planted" in the ground.
- Place the toe of one foot next to the point on the ground.
- Pick up the two tripod legs and look through the optical plummet (OP).
- Move or pivot the instrument until the optical plummet's crosshairs are centered squarely over the point and "plant" the two legs in the ground.
- Using the circular level vial on the instrument base, roughly level the instrument by adjusting the tripod legs.

4.3 Leveling

Leveling is the precise vertical alignment of the instrument's vertical axis.

- Loosen the upper plate clamp.
- Rotate the alidade to position the plate level to a point parallel to any two of the leveling screws.
- Use these two screws to move the bubble to the center of the level.
- Rotate the alidade approximately 90° and adjust the **third screw <u>only</u>** to move the bubble to the center of the level.
- Repeat these two leveling procedures until the bubble remains centered.
- Look through the optical plummet to ensure the instrument is still over the point.
- If not exactly centered over the point, lightly loosen the tripod mounting screw.
- Slide the instrument over the tripod head until centered over the point.
- Re-tighten the tripod mounting screw.
- Check the level of the instrument and re-level if necessary.
- Power on the instrument.

4.4 Sighting to a Prism Reflector

Sighting refers to the aiming of the telescope at the target, bringing the target into focus and aligning the target with the center crosshairs of the reticle.

- Ensure the reticle cross hairs are sharply focused.
    - Point the telescope to a blank area.
    - Look through the eyepiece and rotate the diopter ring until the reticle crosshairs are in sharp focus.
- Use the optical sight to roughly point the telescope to the target.
- Tighten the horizontal plate clamp.
- Look through the eyepiece and move the telescope vertically until the target is in view.
- Tighten the vertical clamp.
- While looking through the telescope, use the horizontal and vertical tangent screws to sight the telescope crosshairs on the center of the prism reflector.
- Rotate the focusing ring to bring the target into sharp focus on the reticle crosshairs.
- Use the horizontal and vertical tangent screws to fine tune the sight to the target.

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5.0 Configuring the Electronic Total Station

Instrument Configuration

All Nikon Total Stations have an **Initial Mode Set** or **Instrument Settings** function. The Initial Mode Set function in the total station is accessed by;

1. Press **[Menu]** key.

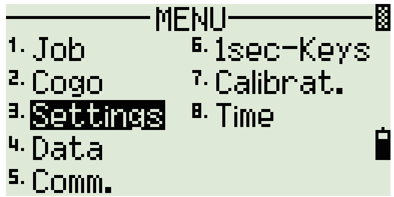

Fig. 5.1: Main Menu Screen

2. Select **3.Settings** by pressing ③ key.

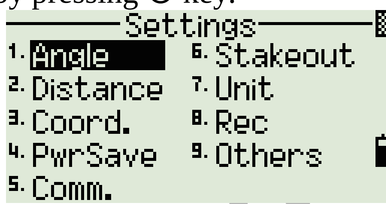

Fig. 5.2: Settings Menu Screen

These initial mode settings are extremely important as they determine the type of data being measured, any corrections being applied and communications to external storage devices. The following settings are the recommended defaults. To toggle between the available selections use the left or right arrow keys ◀/ ▶.

5.1 Settings

1. Angle

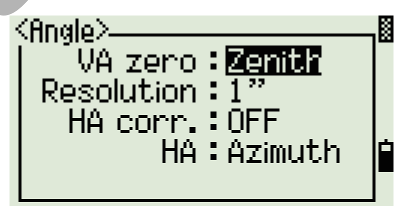

Fig. 5.3: Angle Settings Screen

2. Distance

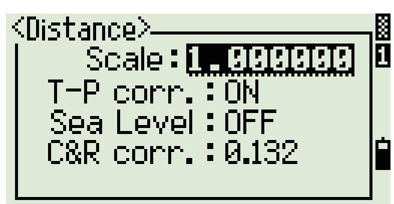

Fig. 5.4: Distance Settings Screen

3. Coord.

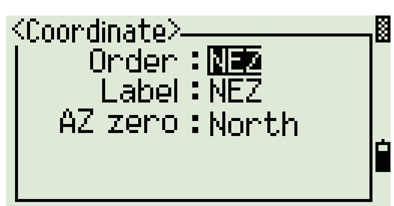

Fig. 5.5: Coordinate Settings Screen

5.1 Settings (cont'd)

4. PwrSave

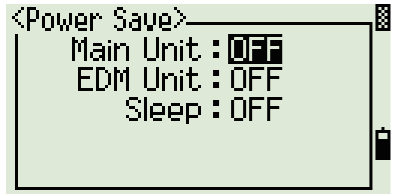

Fig. 5.6: Power Save Settings Screen

5. Comm.

Fig. 5.7: Communications Settings Screen

6. Stakeout

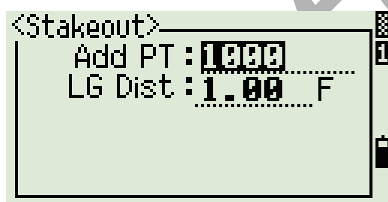

Fig. 5.8: Stakeout Settings Screen

7. Unit

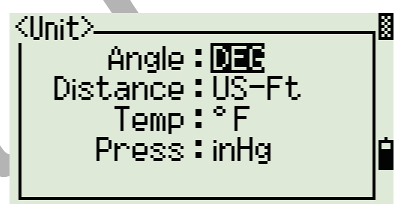

Fig. 5.9: Units Settings Screen

8. Rec

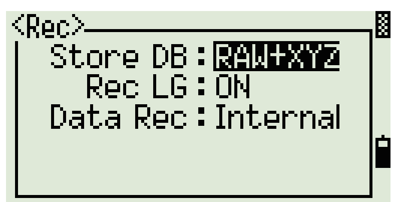

Fig. 5.10: Record Settings Screen

9. Others

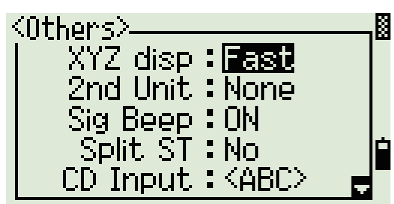

Fig. 5.11: Others Settings Screen

5.2 Prism Constant

The Prism Constant function is accessed by;

1. Press and hold **MSR1** key for one second.

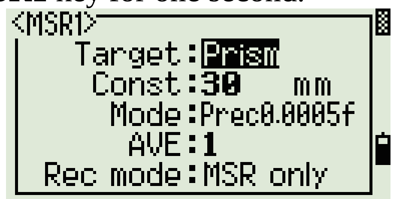

Fig. 5.12: Prism Constant for MSR1 Key

2. Press and hold **MSR2** key for one second.

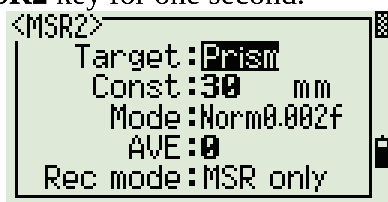

Fig. 5.13: Prism Constant for MSR2 Key

5.3 Height of Target

The Height of Target function in the total station is accessed by;

1. Press **θ HOT** key.

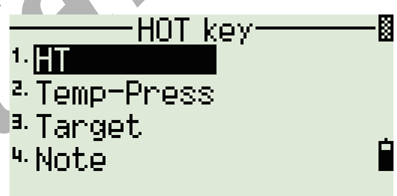

Fig. 5.14: HOT Key Menu Screen

2. Select **1.HT** by pressing ① key and input value for **HT**.

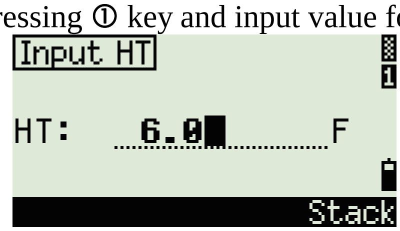

Fig. 5.15: Height of Target Input Screen

5.4 Temperature & Pressure

The Temperature & Pressure function in the total station is accessed by;

1. Press θ **HOT** key.

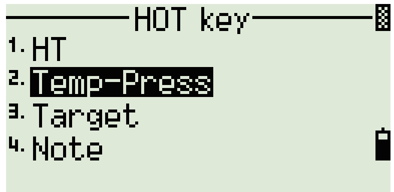

Fig. 5.16: HOT Key Menu Screen

2. Select **2.Temp-Press** by pressing ② key and input values for **Temp** and **Press**.

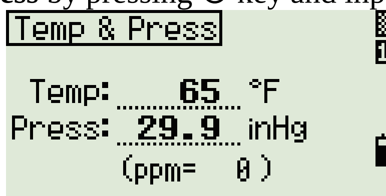

Fig. 5.16: Temperature & Pressure Input Screen

6.0 Data Collection

6.1 Data Collection Surveys

1. Instrument Setup.

- Set the instrument over the control or station point.
- Level the instrument.
- Power on the instrument.
- Tilt the telescope to initialize the Vertical Circle.
- **Basic Measurement Screen (BMS)** is displayed.

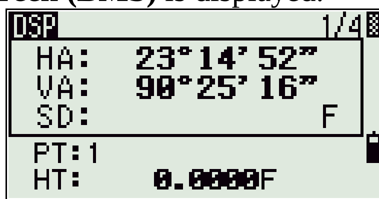

Fig. 6.1: Basic Measurement Screen

2. Create a Job

- Press **[Menu]** key**.**

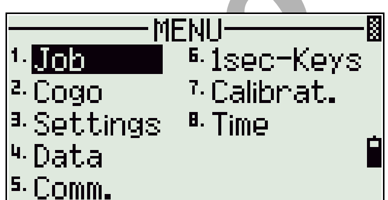

Fig. 6.2: Main Menu Screen

- Select **1.Job** by pressing ① key.

Fig. 6.3: Job Manager Screen

- Select **Creat** softkey by pressing **MSR1** key.

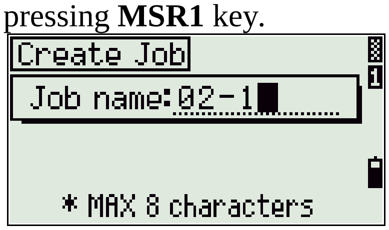

Fig. 6.4: Create Job Screen

- Input **Job Name** up to 8 characters.
- Press **ENT** key.

6.1 Data Collection Surveys (cont'd)

2. Create a Job (cont'd)

- **Create Job Confirmation Screen** is displayed.

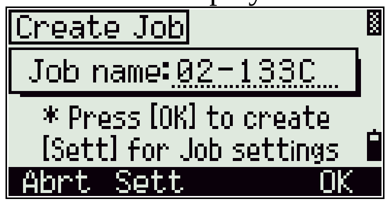

Fig. 6.5: Create Job Confirmation Screen

- Select **OK** softkey by pressing **ANG** key.
- **Basic Measurement Screen (BMS)** is displayed.

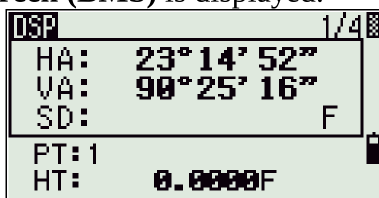

Fig. 6.6: Basic Measurement Screen

3. Station Setup

- Press ⑦ **STN** key**.**

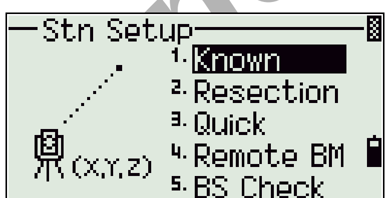

Fig. 6.7: Station Setup Menu Screen

- Select **1.Known** by pressing ① key.

Fig. 6.8: Station Input Screen

- Input Station Point Number **ST**, e.g. **1** and press **ENT**.

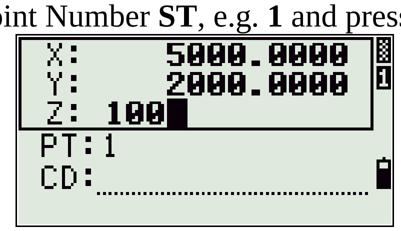

Fig. 6.9: Station Coordinates Input Screen

- Input Station Coordinates **N,E,Z** if displayed as blank and press **ENT**.
- Input Station Code or Description **CD** and press **ENT**.
- Input Station Height of Instrument **HI** and press **ENT**.

6.1 Data Collection Surveys (cont'd)

3. Station Setup (cont'd)

- **Backsight Menu Screen** is displayed.

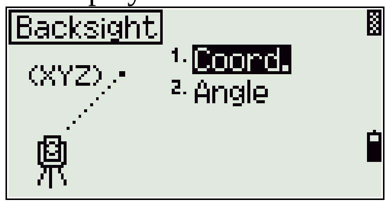

Fig. 6.10: Backsight Menu Screen

- Select either
  - **1.Coord.** if the backsight has known coordinates or
  - **2.Angle** if only the azimuth to the backsight is known.
- Perform backsight observation per prompt sequence below;

Fig. 6.11: Backsight Point Input Screen

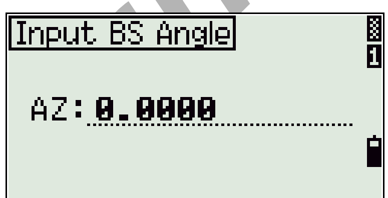

Fig. 6.12: Backsight Azimuth Input Screen

Fig. 6.13: Backsight Measurement Screen

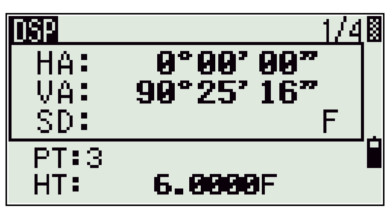

Fig. 6.14: Basic Measurement Screen

6.1 Data Collection Surveys (cont'd)

4. Collection

- Sight the point to be collected.
- Press **MSR1** key.

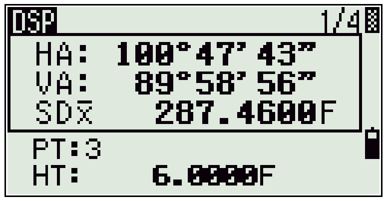

Fig. 6.15: Measurement Screen

- When distance **SD** is displayed, press **REC** key.
- A **"Record PT" Data Input Screen** is displayed.

Fig. 6.16: Record Point Input Screen

  - Input Point Number **PT** if necessary.
  - Input Height of Target **HT** if necessary.
  - Input Code **CD** if necessary by manually typing, or
    - Select from a **List** by pressing **DSP** key, or
    - Select from a **Stack** by pressing **ANG** key.
  - Press **ENT** key to record the point.
- Repeat this sequence for all points to be collected.
- **If moving to new station setup**, measure and record the new Station Point.

6.1 Data Collection Surveys (cont'd)

5. Move to New Station Setup

- Move to new Station.
- Repeat Step 1 above.
- Repeat Step 3 above to **Backsight Menu Screen** is displayed.

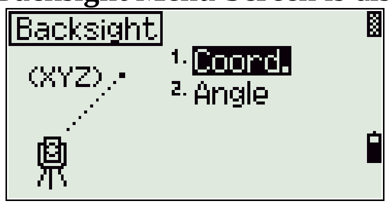

Fig. 6.17: Backsight Menu Screen

- Select **1.Coord.** (the backsight has known coordinates) by pressing ① key.
- Perform backsight observation per prompts.
- Repeat Step 4. as required.

6. Download the Survey Data.

- Prepare computer to accept data.
- Press **[Menu]** key.

Fig. 6.18: Main Menu Screen

- Select **5.Comm.** by pressing ⑤.

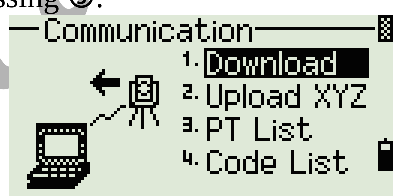

Fig. 6.19: Communications Menu Screen

- Select **1.Download** by pressing ①.
- **Download Settings Screen** is displayed.

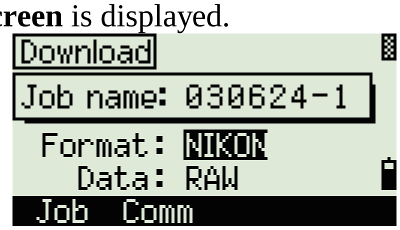

Fig. 6.20: Download Settings Screen

6.1 Data Collection Surveys (cont'd)

- Select
    - **Format: NIKON** and
    - **Data: RAW** and press **ENT** key.

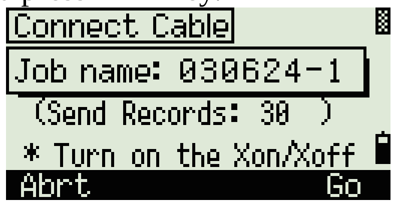

Fig. 6.21: Download Confirmation Screen

- Ensure cable is connected to instrument and computer.
- Select **Go** softkey by pressing **ANG** key.
- **SENDING screen** is displayed with record counter update till Complete.

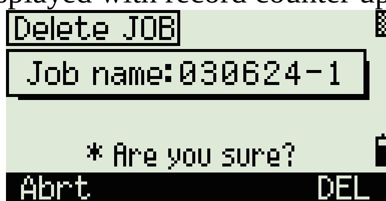

Fig. 6.22: Delete Job Screen

- At the **Delete JOB** screen, press either
    - **Abrt** softkey (**MSR1** key) to **NOT** Delete Job and return to **BMS Screen**, or
    - **DEL** softkey (**ANG** key) to Delete Job and return to **BMS Screen.**

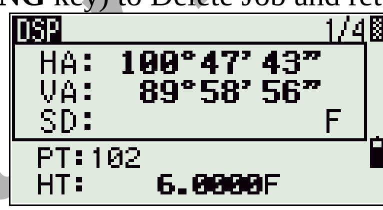

Fig. 6.23: Basic Measurement Screen

7.0 Data Transfer with TransIt Software

TransIt is data transfer software containing functions to transfer data bi-directionally between a Nikon Total Station and a personal computer via the RS-232 serial port and a cable.

Additional functions include the ability to create and edit jobs, download and upload data and export and import survey data between a number of third party data file formats.

7.1 Starting TransIt

- In Windows, double click the **TransIt** icon.
- The **TransIt Main Menu** is displayed.

7.2 TransIt Main Menu

File	Edit	Transfer	Process	Tools	Window	Help

Fig. 7.1: TransIt Main Menu

7.3 TransIt Main Menu Functions

Below is a brief description of each menu item in the **TransIt Main Menu**.

File

- **New Job:** Creates new job.
- **Open Job:** Opens an existing job.
- **Save Job:** Saves a job.
- **Save Job As:** Saves a job to a new location.
- **Import Job:** Imports data from third party data format to Nikon data format.
- **Export Job:** Exports data from Nikon data format to third party data format.
- **Print Report:** Prints Nikon raw and coordinate data file.
- **Properties:** Displays summary of job properties.
- **Exit:** Exits TransIt.

Edit

- **Delete Record:** Deletes current survey record.
- **Undelete Record:** Restores current deleted survey record.
- **Insert Record:** Inserts different types of survey data.
- **Append Record:** Appends survey record to end of current job.
- **Search Record:** Searches for specific survey record.

Transfer

- **Data Recorder to PC:** Transfers survey data from the Data Recorder to the PC.
- **PC to Data Recorder:** Transfers data from the PC to the Data Recor

7.3 TransIt Main Menu Functions (cont'd)

Process

- **Calculate Coords:** Calculates coordinate values.
- **View Reprocess Log:** Displays log record of reprocessing activity.
- **View Upload/Export:** Displays log record of data upload/export activity.

Tools

- **Comm. Settings:** Communications settings for Com Port and Baud Rates.
- **Export Settings:** Export settings for DXF and Coordinate options.
- **Job Settings:** Settings for data type and corrections.
- **Code List Tools:** Tool for creating Code Lists.
- **COGO:** Coordinate Geometry routines.

Window

- **Arrange Icons:** Arrange icons at the bottom of the window.

Help

- **Contents:** Displays TransIt Help Contents.
- **Search for Help On:** Searches for Help on Specific Topics.
- **Technical Support:** Technical Support user & problem information.
- **About:** Displays licensing and software information.

7.4 TransIt Data Downloading from the Total Station

Personal Computer

- Start **TransIt** by double clicking on the **TransIt icon** in Windows.
- Select **Transfer** from the **TransIt Main Menu**.
- Select **Data Recorder to PC** from the **Transfer Menu**.
- Ensure **Data Recorder** selection box displays **DTM-352/332 (or DTM-502)**.
- Enter name of job in the **Job Name** (Jobname.raw) selection box and click on **OK** box.
- At the "Prepare Nikon Total Station...." screen, click on **OK** box.
- At the **"TransIt Transfer Complete"** screen, select **OK**.

DTM-502/352/332 Total Station

To initiate the **Download** from the total station,

- Connect the total station-to-PC serial cable.
- Press **[Menu]** key.

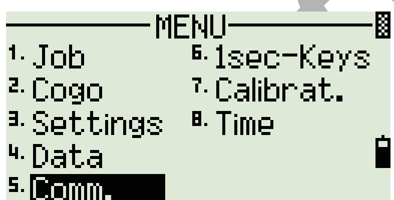

Fig. 7.1: Main Menu Screen

- Select **5.Comm.** by pressing ⑤.

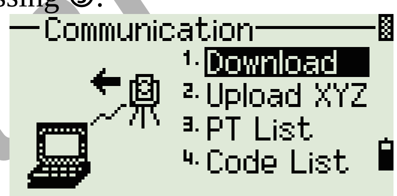

Fig. 7.2: Communications Menu Screen

- Select **1.Download** by pressing ①.
- **Download Settings Screen** is displayed.

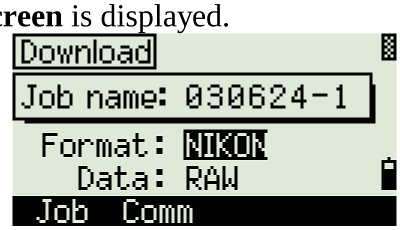

Fig. 7.3: Download Settings Screen

- Select
  - **Format: NIKON** and
  - **Data: RAW** and press **ENT** key.

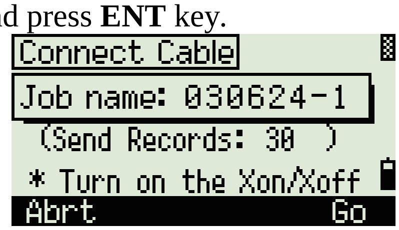

Fig. 7.4: Download Confirmation Screen

7.4 TransIt Data Downloading from the Total Station (cont'd)

- Ensure cable is connected to instrument and computer.
- Select **Go** softkey by pressing **ANG** key.
- **SENDING screen** is displayed with record counter update till Complete.

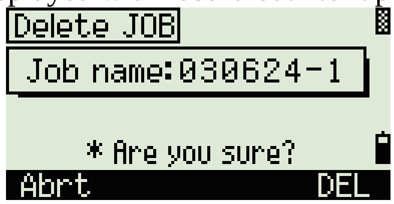

Fig. 7.5: Delete Job Screen

- At the **Delete JOB** screen, press either
  - **Abrt** softkey (**MSR1** key) to **NOT** Delete Job and return to **BMS Screen**, or
  - **DEL** softkey (**ANG** key) to Delete Job and return to **BMS Screen.**

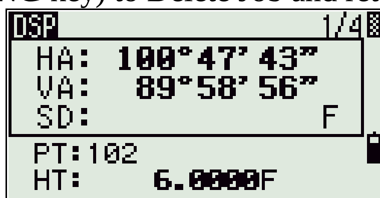

Fig. 7.6: Basic Measurement Screen

- Select **OK** at the PC after the download is complete.

7.5 TransIt Data Conversion

TransIt can **import** a variety of third party software data formats into total station format data and **export** total station format data to a variety of third party software data formats.

To perform a data conversion;

Data Export

To export Nikon data to a third party data format;

- Select **File** from the **TransIt Main Menu**.
- Select **Export Job** from the **File Menu**.
- Select the **Export Format** from the Export Format selection box.
- Verify the name of the job in the **Job Name** selection box and click on **OK** box.
- Type the name of the output file to be created and press **OK**.
- At the **"TransIt Export Complete"** screen, select **OK**.

Data Import

To import a third party data format into the Nikon data format;

- Select **File** from the **TransIt Main Menu**.
- Select **Import Job** from the **File Menu**.
- Select the **Data Format** from the Data Format selection box.
- Select the name of the job in the **Job Name** selection box and click on **OK** box.
- At the **"TransIt Import Complete"** screen, select **OK**.

7.6 TransIt Data Uploading to the Total Station

Personal Computer

- Start **TransIt** by double clicking on the **TransIt icon** in Windows.
- Select **File** from the **TransIt Main Menu**.
- Select **Import Job** from the **File Menu**.
- Select the **Data Format** from the Data Format selection box.
- Select the import job name in the **Job Name** selection box and click on **OK** box.
- At the **"TransIt Import Complete"** screen, select **OK**.
- Select **Transfer** from the **TransIt Main Menu**.
- Select **PC to Data Recorder** from the **Transfer Menu**.
- Select **DTM-502/352/332** from Data Recorder selection box and click **OK**.
- Enter the name of the job in the **Job Name** selection box and click on **OK** box.
- Prepare total station to accept the data to be uploaded.
- At the **"TransIt Information"** screen, select **OK**.

DTM-502/352/332 Total Station

To prepare the total station to accept the Uploaded data,

- Connect the total station-to-PC serial cable.
- **Create a Job** to receive the data.
    - Press **[Menu]** key**.**

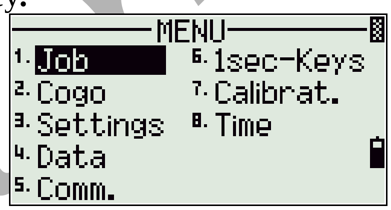

Fig. 7.7: Main Menu Screen

    - Select **1.Job** by pressing ① key.

Fig. 7.8: Job Manager Screen

    - Select **Creat** softkey by pressing **MSR1** key.

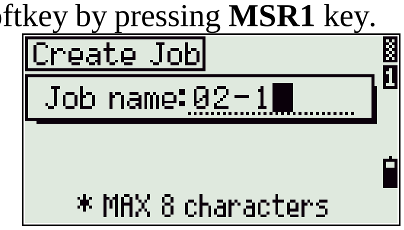

Fig. 7.9: Create Job Screen

        - Input **Job Name** up to 8 characters.
        - Press **ENT** key.

7.6 TransIt Data Uploading to the Total Station (cont'd)

- **Create Job Confirmation Screen** is displayed.

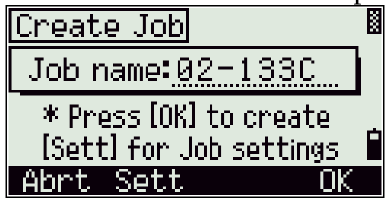

Fig. 7.10: Create Job Confirmation Screen

- Select **OK** softkey by pressing **ANG** key.
- **Basic Measurement Screen (BMS)** is displayed.

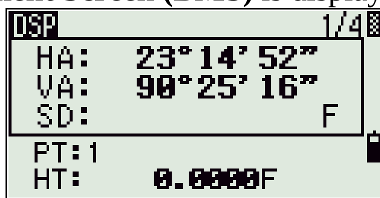

Fig. 7.11: Basic Measurement Screen

- Press **[Menu]** key.

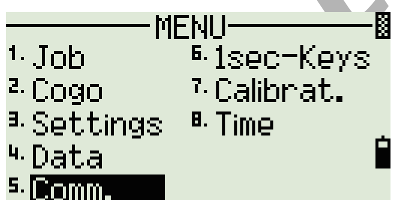

Fig. 7.12: Main Menu Screen

- Select **5.Comm.** by pressing ⑤.

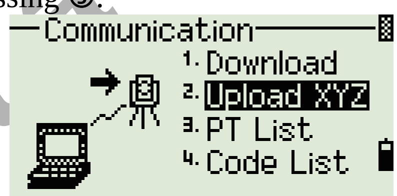

Fig. 7.13: Communications Menu Screen

- Select **2.Upload XYZ** by pressing ②.
- **Upload Format Screen** is displayed.

Fig. 7.14: Upload Format Screen

- Ensure cable is connected to instrument and computer and press **ENT** key.
- **Upload Confirmation Screen** is displayed.

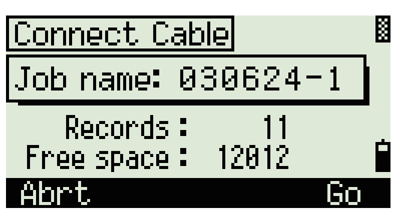

Fig. 7.15: Upload Confirmation Screen

7.6 TransIt Data Uploading to the Total Station (cont'd)

- Select **Go** softkey by pressing **ANG** key.
- **RECEIVING screen** is displayed with record counter update till Complete.
  - At the PC, select **OK** to initiate the data transfer.
- The **Basic Measurement Screen (BMS)** is displayed.

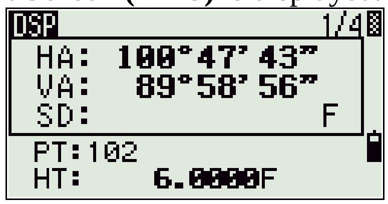

Fig. 7.16: Basic Measurement Screen

8.0 Stakeout Survey from a Known Point

8.1 Stakeout from a Known Point

Points to be staked out may be uploaded or manually input into the total station.

1. Input Stakeout Points - Upload Coordinates.

Upload Coordinates.

Personal Computer

- Start **TransIt** by double clicking on the **TransIt icon** in Windows.
- Select **Transfer** from the **TransIt Main Menu**.
- Select **PC to Data Recorder** from the **Transfer Menu**.
- Select **DTM-502/352/332** from Data Recorder selection box and click **OK**.
- Enter the name of the job in the Job Name selection box and click on **OK** box.
- Prepare the total station to accept the data to be uploaded.
- At the "TransIt Information" screen, select **OK**.

DTM-502/352/332 Total Station

To prepare the total station to accept the Uploaded data,

- Connect the total station-to-PC serial cable.
- **Create a Job** to receive the data.
    - Press **[Menu] key.**

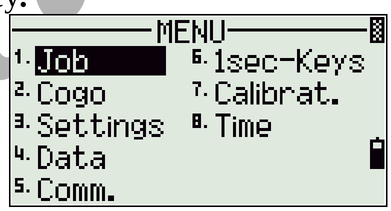

Fig. 8.1: Main Menu Screen

- Select **1.Job** by pressing ① key.

Fig. 8.2: Job Manager Screen

- Select **Creat** softkey by pressing **MSR1** key.

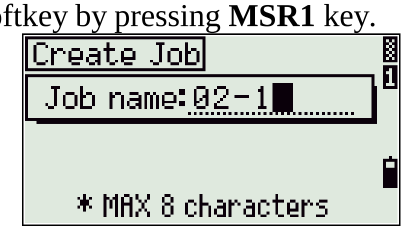

Fig. 8.3: Create Job Screen

- Input **Job Name** up to 8 characters and press **ENT** key.

1. Input Stakeout Points - Upload Coordinates. (cont'd)

- **Create Job Confirmation Screen** is displayed.

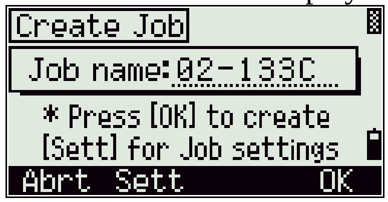

Fig. 8.4: Create Job Confirmation Screen

- Select **OK** softkey by pressing **ANG** key.
- **Basic Measurement Screen (BMS)** is displayed.

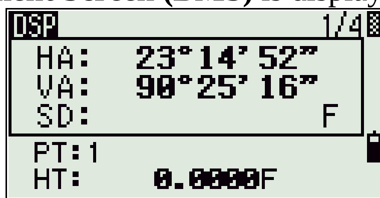

Fig. 8.5: Basic Measurement Screen

- Press **[Menu]** key.

Fig. 8.6: Main Menu Screen

- Select **5.Comm.** by pressing ⑤.

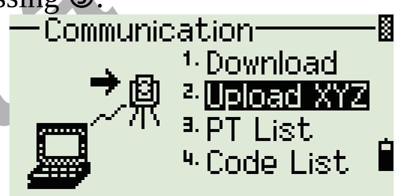

Fig. 8.7: Communications Menu Screen

- Select **2.Upload XYZ** by pressing ②.
- **Upload Format Screen** is displayed.

Fig. 8.8: Upload Format Screen

- Ensure cable is connected to instrument and press **ENT** key.
- **Upload Confirmation Screen** is displayed.

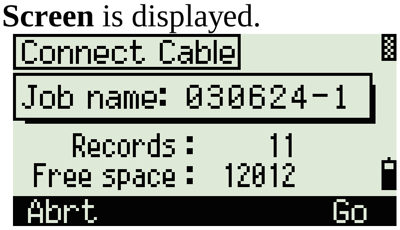

Fig. 8.9: Upload Confirmation Screen

- Select **Go** softkey by pressing **ANG** key.

1. Input Stakeout Points - Upload Coordinates. (cont'd)

- **RECEIVING screen** is displayed with record counter update till Complete.
    - At the PC, select **OK** to initiate the data transfer.
- The **Basic Measurement Screen (BMS)** is displayed.

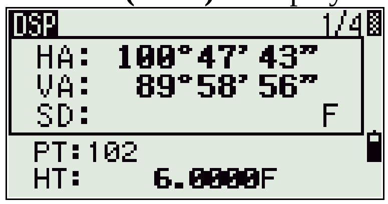

Fig. 8.10: Basic Measurement Screen

8.1 Stakeout from a Known Point (cont'd)

1. Input Stakeout Points - Manual Input.

Manually Input Coordinates.

- **Create a Job** to input the data.
  - Press **[Menu]** key**.**

Fig. 8.11: Main Menu Screen

  - Select **1.Job** by pressing ① key.

Fig. 8.12: Job Manager Screen

  - Select **Creat** softkey by pressing **MSR1** key.

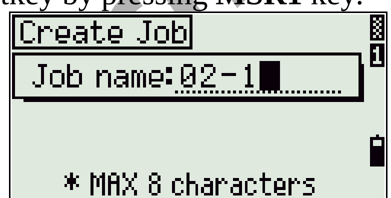

Fig. 8.13: Create Job Screen

    - Input **Job Name** up to 8 characters and press **ENT** key.
  - **Create Job Confirmation Screen** is displayed.

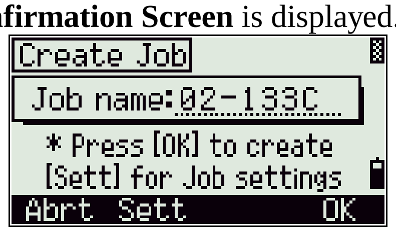

Fig. 8.14: Create Job Confirmation Screen

  - Select **OK** softkey by pressing **ANG** key.
  - **Basic Measurement Screen (BMS)** is displayed.

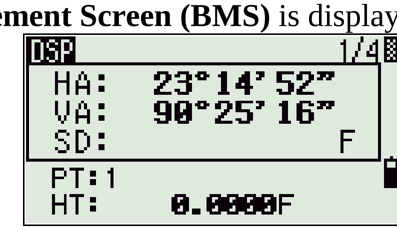

Fig. 8.15: Basic Measurement Screen

8.1 Stakeout from a Known Point (cont'd)

1. Input Stakeout Points - Manual Input. (Cont'd)

- Press **[Menu]** key.

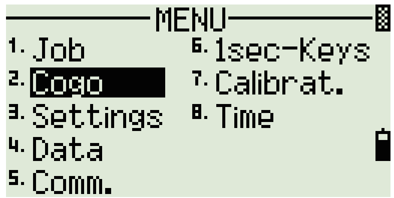

Fig. 8.16: Main Menu Screen

- Select **2.Cogo** by pressing ②.

Fig. 8.17: Cogo Menu Screen

- Select **5.Input XYZ** by pressing ⑤.
- A **Coordinate Input Screen** is displayed.

Fig. 8.18: Coordinate Input Screen

- Input Northing **N:** and press **ENT** key.
- Input Easting **E:** and press **ENT** key.
- Input Elevation **Z:** if necessary and press **ENT** key.
- Input Point Number **PT** if necessary.
- Input Code **CD** if necessary.
- Press **ENT** key to record the point.
- Repeat **Coordinate Input** sequence for all points to be input.
- Press **ESC** key three times to return to **Basic Measurement Screen**.

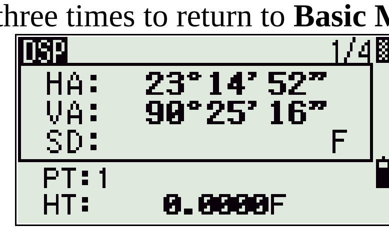

Fig. 8.19: Basic Measurement Screen

2. Instrument Setup.

- Set the instrument over the control or station point.
- Level the instrument.
- Power on the instrument.
- Tilt the telescope to initialize the Vertical Circle.
- **Basic Measurement Screen (BMS)** is displayed.

8.1 Stakeout from a Known Point (cont'd)

3. Station Setup

- Press ⑦ **STN** key.

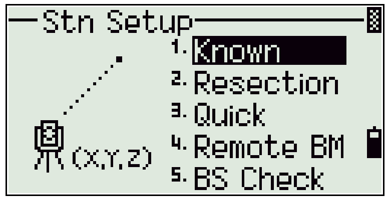

Fig. 8.20: Station Setup Menu Screen

- Select **1.Known** by pressing ① key.

Fig. 8.21: Station Input Screen

- Input Station Point Number **ST**, e.g. **1** and press **ENT**.

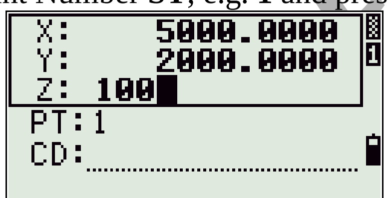

Fig. 8.22: Station Coordinates Input Screen

- Input Station Coordinates **N,E,Z** if displayed as blank and press **ENT**.
- Input Station Code or Description **CD** and press **ENT**.
- Input Station Height of Instrument **HI** and press **ENT**.

- **Backsight Menu Screen** is displayed.

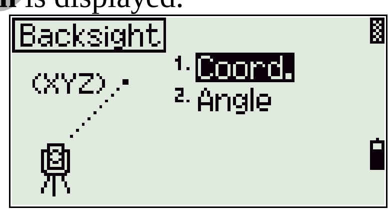

Fig. 8.23: Backsight Menu Screen

- Select either
  - **1.Coord.** if the backsight has known coordinates or
  - **2.Angle** if only the azimuth to the backsight is known.
- Perform backsight observation per prompt sequence below;

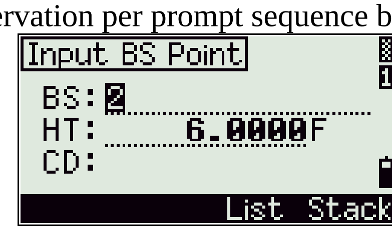

Fig. 8.24: Backsight Point Input Screen

8.1 Stakeout from a Known Point (cont'd)

3. Station Setup (cont'd)

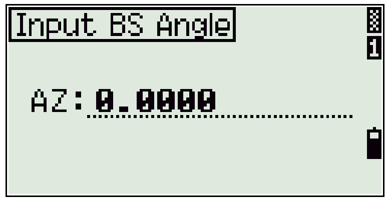

Fig. 8.25: Backsight Azimuth Input Screen

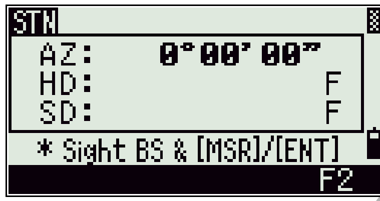

Fig. 8.26: Backsight Measurement Screen

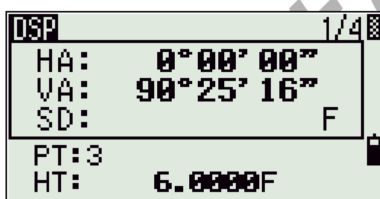

Fig. 8.27: Basic Measurement Screen

4. Stakeout

- Press ⑧ **S-O** key.

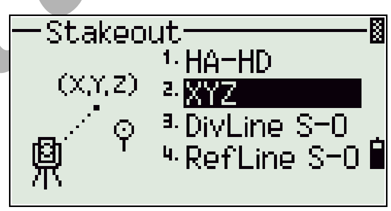

Fig. 8.28: Stakeout Menu Screen

- Select **2.XYZ** by pressing ② key.
- The **Stakeout Point Input Screen** is displayed.

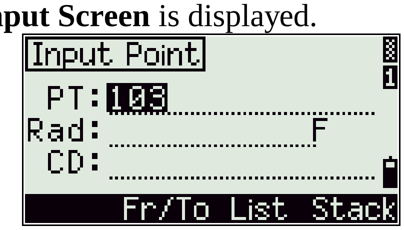

Fig. 8.29: Stakeout Point Input Screen

- Input **Stakeout Point Number PT:** of point to be staked.
- The coordinates of the point to be staked should be displayed.
    - Input Point Coordinates **N,E,Z** if displayed as blank.
        - Input Northing **N:** and press **ENT** key.
        - Input Easting **E:** and press **ENT** key.
        - Input Elevation **Z:** if necessary.
- Press **ENT** key to accept the point.

8.1 Stakeout from a Known Point (cont'd)

4. Stakeout (cont'd)

- The **Stakeout Observation Screen** showing **dHA** and **HD** is displayed.
  - **dHA:** is the zero countdown horizontal angle (turn in direction of arrow).
  - **HD:** is the horizontal distance to the stakeout point.

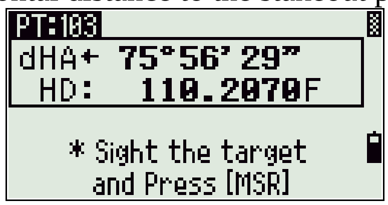

Fig. 8.30: Stakeout Observation Screen

- Rotate the instrument until **dHA** is **0°00'00"**. You are now "on-line".
- Sight the target position on-line and press **MSR1 or MSR2** key.
- The distance the prism is to be moved **In/Out** to be on the stakeout point is displayed.

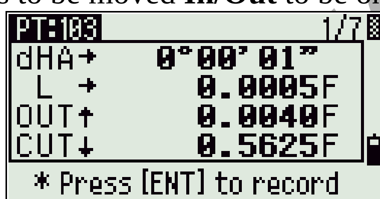

Fig. 8.31: In/Out Stakeout Screen

- Move the target and repeat measurements until **In/Out** distance is **0.000**.
- To record the coordinates of the target position
  - Press **REC** key to display the coordinates, point number and code.

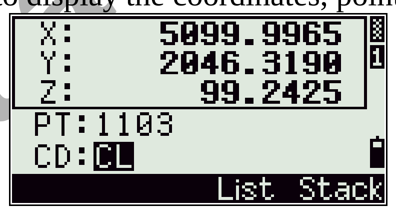

Fig. 8.32: Coordinate Display Screen

    - Change Point Number if necessary.
    - Enter Code if necessary.
  - Press **ENT** key to record the staked point.
- The **Stakeout Observation Screen** showing **dHA** and **HD** is displayed.
- Press **ESC** key to return to the **Stakeout Point Input Screen**.
- The **Stakeout Point Number PT:** of the next point to be staked is displayed.

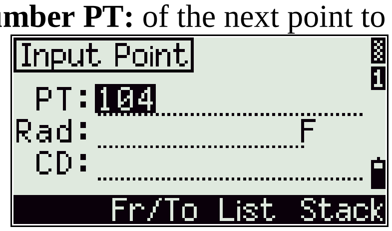

Fig. 8.33: Stakeout Point Input Screen

- Repeat until all points have been staked.

8.1 Stakeout from a Known Point (cont'd)

5. Download the Stakeout Data.

- Prepare computer to accept data.
- Press **[Menu]** key.

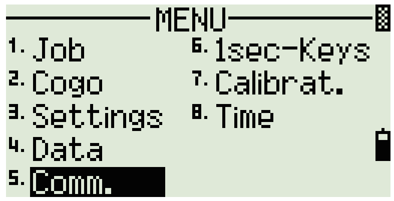

Fig. 8.34: Main Menu Screen

- Select **5.Comm.** by pressing ⑤.

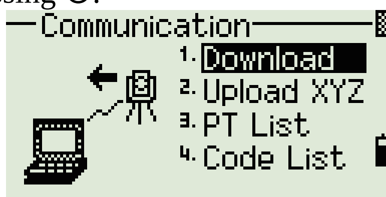

Fig. 8.35: Communications Menu Screen

- Select **1.Download** by pressing ①.
- **Download Settings Screen** is displayed.

Fig. 8.36: Download Settings Screen

- Select
  - **Format: NIKON** and
  - **Data: RAW** and press **ENT** key.

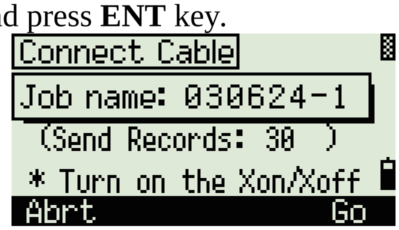

Fig. 8.37: Download Confirmation Screen

- Ensure cable is connected to instrument and computer.
- Select **Go** softkey by pressing **ANG** key.
- **SENDING screen** is displayed with record counter update till Complete.

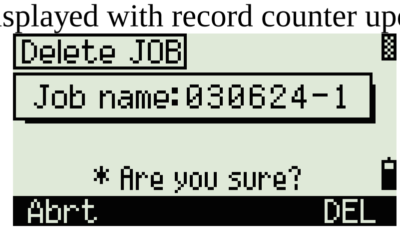

Fig. 8.38: Delete Job Screen

- At the **Delete JOB** screen, press either
  - **Abrt** softkey (**MSR1** key) to <u>**NOT**</u> Delete Job and return to **BMS Screen**, or
  - **DEL** softkey (**ANG** key) to Delete Job and return to **BMS Screen.**

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9.0 Stakeout Survey from an Unknown Point

9.1 Stakeout from an Unknown Point

Points to be staked out may be uploaded or manually input into the total station.

1. Input Stakeout Points - Upload Coordinates.

Upload Coordinates.

Personal Computer

- Start **TransIt** by double clicking on the **TransIt icon** in Windows.
- Select **Transfer** from the **TransIt Main Menu**.
- Select **PC to Data Recorder** from the **Transfer Menu**.
- Select **DTM-502/352/332** from Data Recorder selection box and click **OK**.
- Enter the name of the job in the Job Name selection box and click on **OK** box.
- Prepare the total station to accept the data to be uploaded.
- At the "TransIt Information" screen, select **OK**.

DTM-502/352/332 Total Station

To prepare the total station to accept the Uploaded data,
- Connect the total station-to-PC serial cable.
- **Create a Job** to receive the data.
  - Press **[Menu] key.**

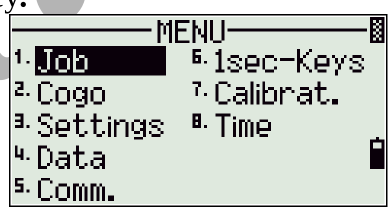

Fig. 9.1: Main Menu Screen

  - Select **1.Job** by pressing ① key.

Fig. 9.2: Job Manager Screen

  - Select **Creat** softkey by pressing **MSR1** key.

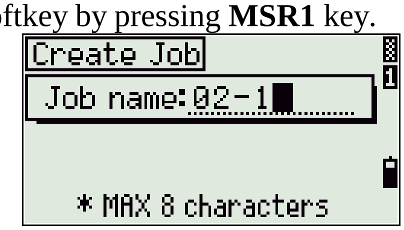

Fig. 9.3: Create Job Screen

  - Input **Job Name** up to 8 characters and press **ENT** key.

9.1 Stakeout from an Unknown Point (cont'd)

1. Input Stakeout Points - Upload Coordinates. (cont'd)

- **Create Job Confirmation Screen** is displayed.

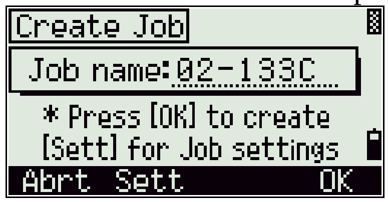

Fig. 9.4: Create Job Confirmation Screen

- Select **OK** softkey by pressing **ANG** key.
- **Basic Measurement Screen (BMS)** is displayed.

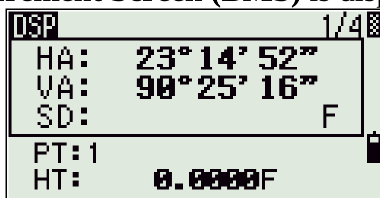

Fig. 9.5: Basic Measurement Screen

- Press **[Menu]** key.

Fig. 9.6: Main Menu Screen

- Select **5.Comm.** by pressing ⑤.

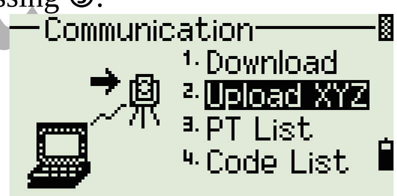

Fig. 9.7: Communications Menu Screen

- Select **2.Upload XYZ** by pressing ②.
- **Upload Format Screen** is displayed.

Fig. 9.8: Upload Format Screen

- Ensure cable is connected to instrument and computer and press **ENT** key.

9.1 Stakeout from an Unknown Point (cont'd)

1. Input Stakeout Points - Upload Coordinates. (cont'd)

- **Upload Confirmation Screen** is displayed.

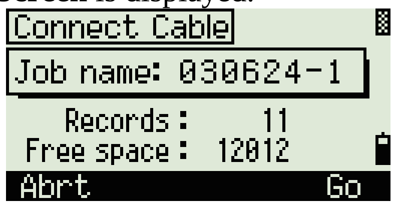

Fig. 9.9: Upload Confirmation Screen

- Select **Go** softkey by pressing **ANG** key.
- **RECEIVING screen** is displayed with record counter update till Complete.
    - At the PC, select **OK** to initiate the data transfer.
- The **Basic Measurement Screen (BMS)** is displayed.

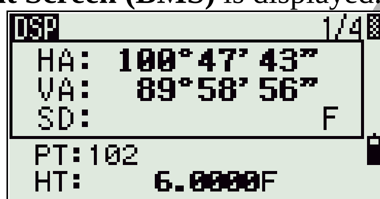

Fig. 9.10: Basic Measurement Screen

9.1 Stakeout from an Unknown Point (cont'd)

1. Input Stakeout Points - Manual Input.

Manually Input Coordinates.

- **Create a Job** to input the data.
  - Press **[Menu]** key**.**

Fig. 9.11: Main Menu Screen

- Select **1.Job** by pressing ① key.

Fig. 9.12: Job Manager Screen

- Select **Creat** softkey by pressing **MSR1** key.

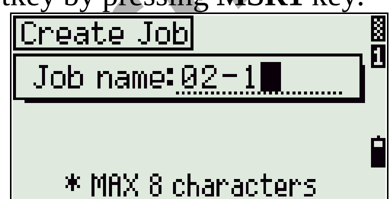

Fig. 9.13: Create Job Screen

- Input **Job Name** up to 8 characters and press **ENT** key.
- **Create Job Confirmation Screen** is displayed.

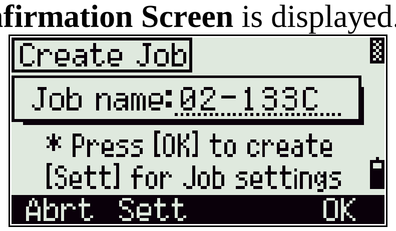

Fig. 9.14: Create Job Confirmation Screen

- Select **OK** softkey by pressing **ANG** key.
- **Basic Measurement Screen (BMS)** is displayed.

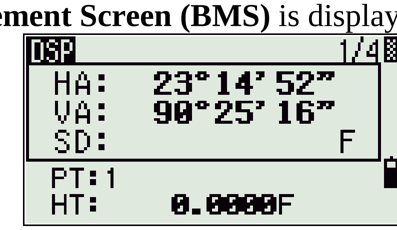

Fig. 9.15: Basic Measurement Screen

9.1 Stakeout from an Unknown Point (cont'd)

1. Input Stakeout Points - Manual Input. (Cont'd)

- Press **[Menu]** key.

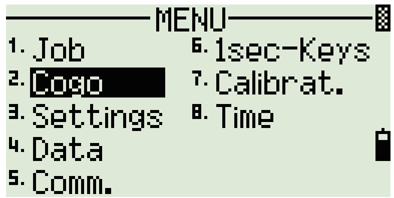

Fig. 9.16: Main Menu Screen

- Select **2.Cogo** by pressing ②.

Fig. 9.17: Cogo Menu Screen

- Select **5.Input XYZ** by pressing ⑤.
- A **Coordinate Input Screen** is displayed.

Fig. 9.18: Coordinate Input Screen

  - Input Northing **N:** and press **ENT** key.
  - Input Easting **E:** and press **ENT** key.
  - Input Elevation **Z:** if necessary and press **ENT** key.
  - Input Point Number **PT** if necessary.
  - Input Code **CD** if necessary.
  - Press **ENT** key to record the point.
- Repeat **Coordinate Input** sequence for all points to be input.
- Press **ESC** key three times to return to **Basic Measurement Screen**.

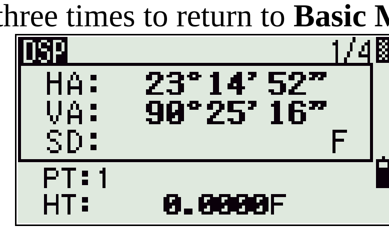

Fig. 9.19: Basic Measurement Screen

2. Instrument Setup.

- Set the instrument over the control or station point.
- Level the instrument.
- Power on the instrument.
- Tilt the telescope to initialize the Vertical Circle.
- **Basic Measurement Screen (BMS)** is displayed.

9.1 Stakeout from an Unknown Point (cont'd)

3. Station Setup

When using resection techniques (measuring to two or more known points to compute coordinates for an unknown point), it is recommended measurements be taken to a minimum three known points. If only two known points are available, the angle of intersection at the unknown point should be "strong" – that is, not too flat nor too sharp but ideally somewhere around 90°.

- Press ⑦ **STN** key**.**

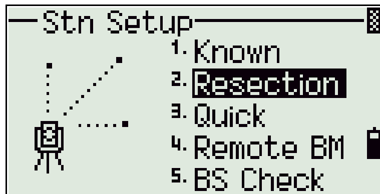

Fig. 9.20: Station Setup Menu Screen

- Select **2.Resection** by pressing ② key.

Fig. 9.21: Point Input Screen (PT1)

- Input First Known Point Number **PT1**, e.g. **101** and press **ENT**.
- Input Coordinates **N,E,Z** of PT1 if displayed as blank and press **ENT**.
- Input Height of Target **HT** at PT1 and press **ENT**.
- Input Code or Description **CD** of PT1 and press **ENT**.

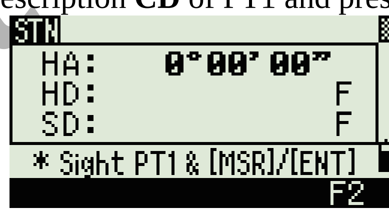

Fig. 9.22: Measure PT1 Instruction Screen

- Sight PT1 and press **MSR1** key.

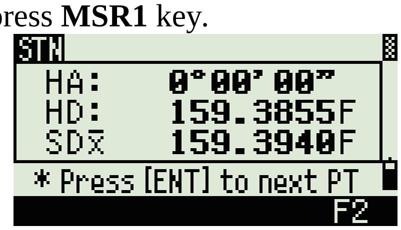

Fig. 9.23: Measurement Data to PT1

- Measurement data to PT1 is displayed. Press **ENT**.
- Input Second Known Point Number **PT2**, e.g. **2** and press **ENT**.

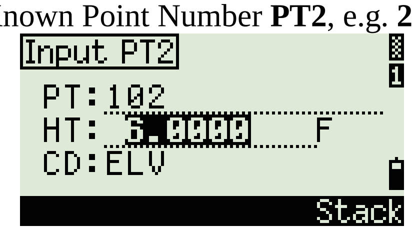

Fig. 9.24: Point Input Screen (PT2)

9.1 Stakeout from an Unknown Point (cont'd)

3. Station Setup (cont'd)

- Input Coordinates **N,E,Z** of PT2 if displayed as blank and press **ENT**.
- Input Height of Target **HT** at PT2 and press **ENT**.
- Input Code or Description **CD** of PT2 and press **ENT**.

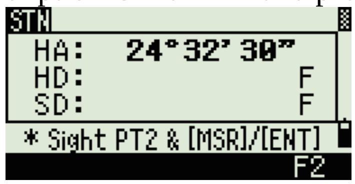

Fig. 9.25: Measure PT2 Instruction Screen

- Sight PT2 and press **MSR1** key.

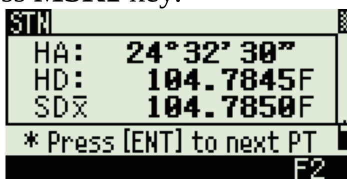

Fig. 9.26: Measurement Data to PT2

- Measurement data to PT2 is displayed. Press **ENT**.

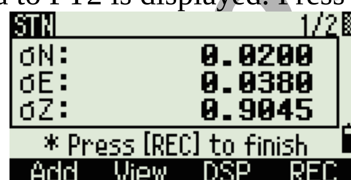

Fig. 9.27: Standard Deviations Screen

- Standard Deviations for the computed coordinates are displayed.
- Press **DSP** softkey (**DSP** key).

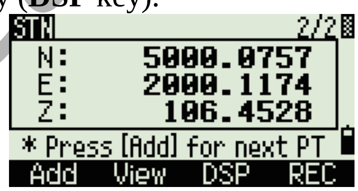

Fig. 9.28: Computed Coordinates Screen

- Computed Coordinates for the Unknown Point Number **N,E,Z** are displayed.
- Press **Add** softkey (**MSR1** key) to add another known point measurement, or
- Press **REC** softkey (**ANG** key) or **ENT** key to record the station.

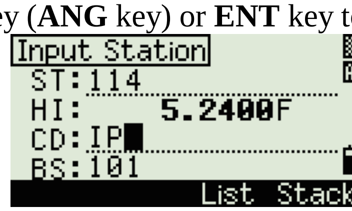

Fig. 9.29: Station Input Screen

- At the **Station Input** screen,
  - Input Station Point Number **ST** of the Unknown Point and press **ENT**.
  - Input Height of Instrument **HI** and press **ENT**.
  - Input Code or Description **CD** and press **ENT**.
  - The Backsight Point **BS** is defaulted to the first measured point PT1.
- Press **ENT** key to complete Station Setup by Resection.
- **Basic Measurement Screen** is displayed.

9.1 Stakeout from an Unknown Point (cont'd)

4. Stakeout

- Press ⑧ **S-O** key**.**

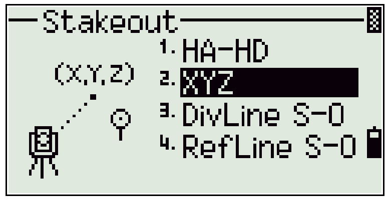

Fig. 9.30: Stakeout Menu Screen

- Select **2.XYZ** by pressing ② key**.**
- The **Stakeout Point Input Screen** is displayed.

Fig. 9.31: Stakeout Point Input Screen

- Input **Stakeout Point Number PT:** of point to be staked.
- The coordinates of the point to be staked should be displayed.
  - Input Point Coordinates **N,E,Z** if displayed as blank.
    - Input Northing **N:** and press **ENT** key.
    - Input Easting **E:** and press **ENT** key.
    - Input Elevation **Z:** if necessary.
- Press **ENT** key to accept the point.
- The **Stakeout Observation Screen** showing **dHA** and **HD** is displayed.
  - **dHA:** is the zero countdown horizontal angle (turn in direction of arrow).
  - **HD:** is the horizontal distance to the stakeout point.

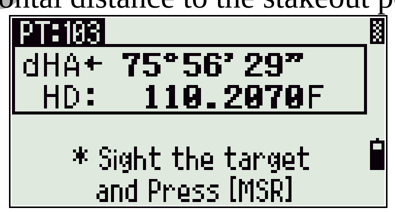

Fig. 9.32: Stakeout Observation Screen

- Rotate the instrument until **dHA** is **0°00'00"**. You are now "on-line".
- Sight the target position on-line and press **MSR1 or MSR2** key.
- The distance the prism is to be moved **In/Out** to be on the stakeout point is displayed.

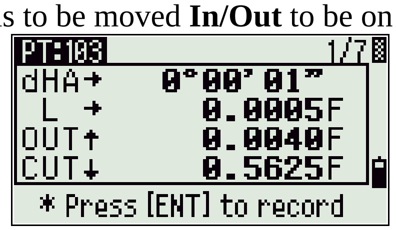

Fig. 9.33: In/Out Stakeout Screen

- Move the target and repeat measurements until **In/Out** distance is **0.000**.
- To record the coordinates of the target position
  - Press **REC** key to display the coordinates, point number and code.

9.1 Stakeout from an Unknown Point (cont'd)

4. Stakeout (cont'd)

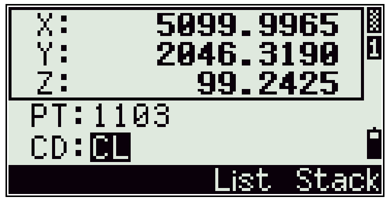

Fig. 9.34: Coordinate Display Screen

- Change Point Number if necessary.
- Enter Code if necessary.
- Press **ENT** key to record the staked point.
- The **Stakeout Observation Screen** showing **dHA** and **HD** is displayed.
- Press **ESC** key to return to the **Stakeout Point Input Screen**.
- The **Stakeout Point Number PT:** of the next point to be staked is displayed.

Fig. 9.35: Stakeout Point Input Screen

- Repeat until all points have been staked.

5. Download the Stakeout Data.

- Prepare computer to accept data.
- Press **[Menu]** key.

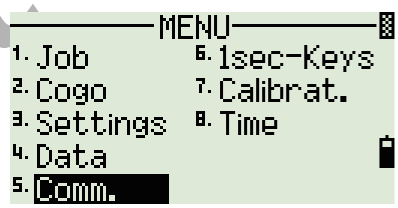

Fig. 9.36: Main Menu Screen

- Select **5.Comm.** by pressing ⑤.

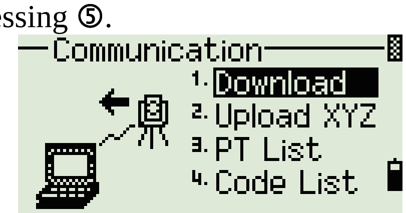

Fig. 9.37: Communications Menu Screen

- Select **1.Download** by pressing ①.
- **Download Settings Screen** is displayed.

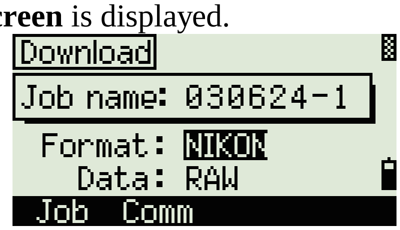

Fig. 9.38: Download Settings Screen

9.1 Stakeout from an Unknown Point (cont'd)

4. Stakeout (cont'd)

- Select
  - **Format: NIKON** and
  - **Data: RAW** and press **ENT** key.

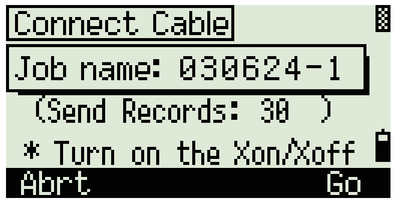

Fig. 9.39: Download Confirmation Screen

- Ensure cable is connected to instrument and computer.
- Select **Go** softkey by pressing **ANG** key.
- **SENDING screen** is displayed with record counter update till Complete.

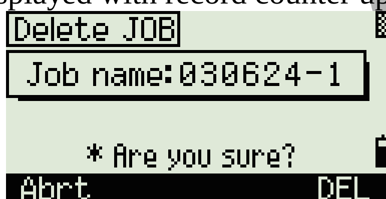

Fig. 9.40: Delete Job Screen

- At the **Delete JOB** screen, press either
  - **Abrt** softkey (**MSR1** key) to **NOT** Delete Job and return to **BMS Screen**, or
  - **DEL** softkey (**ANG** key) to Delete Job and return to **BMS Screen.**

10.0 Resetting Missing Points Using Stakeout

10.1 Resetting Missing Points

This section assumes the Missing Points to be “reset” or “restaked” are contained in an existing job on the total station.

1. Open or Activate the Existing Job.

- **Open the Job** containing the points to be reset.
  - Press **[Menu]** key.

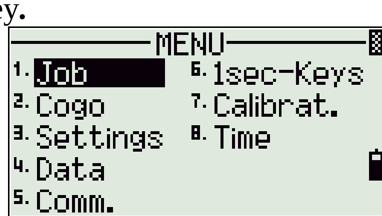

Fig. 10.1: Main Menu Screen

  - Select **1.Job** by pressing ① key.
  - The **Job Manager** screen is displayed.

Fig. 10.2: Job Manager Screen

  - Move the **Highlight Bar** to the correct job using the ▲/▼ keys.
  - Press **ENT** key to open the correct job.
  - **Basic Measurement Screen (BMS)** is displayed.

2. Instrument Setup.

- Set the instrument over an existing known point.
- Level the instrument.
- Power on the instrument.
- Tilt the telescope to initialize the Vertical Circle.
- **Basic Measurement Screen (BMS)** is displayed.

10.1 Resetting Missing Points (cont'd)

3. Station Setup

- Press ⑦ **STN** key.

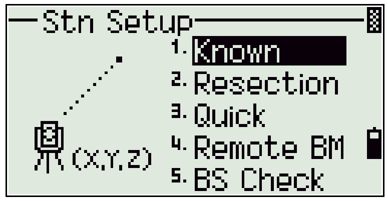

Fig. 10.3: Station Setup Menu Screen

- Select **1.Known** by pressing ① key.

Fig. 10.4: Station Input Screen

  - Input Station Point Number **ST**, e.g. **1** and press **ENT**.

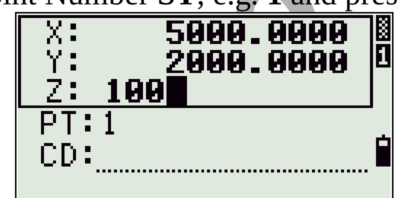

Fig. 10.5: Station Coordinates Input Screen

  - The Station Coordinates **N,E,Z** are displayed..
  - Input Station Code or Description **CD** and press **ENT**.
  - Input Station Height of Instrument **HI** and press **ENT**.
- **Backsight Menu Screen** is displayed.

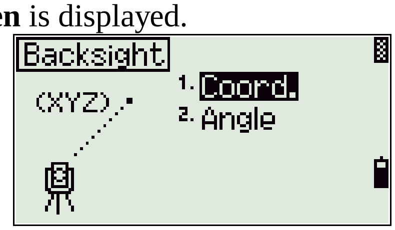

Fig. 10.4: Backsight Menu Screen

- Select **1.Coord.** by pressing ① key.
  - Input Backsight Point Number **BS**, e.g. **12** and press **ENT**.
  - The Backsight Point Coordinates **N,E,Z** are displayed.
  - Input Height of Instrument **HI** at the Backsight Point and press **ENT**.
  - Input Code or Description **CD** and press **ENT**.
- Sight Backsight Point **BS** and press **MSR1** key.

.
- Press **ENT** key to complete Station Setup
- **Basic Measurement Screen (BMS)** is displayed.

10.1 Resetting Missing Points (cont'd)

4. Stakeout

- Press ⑧ **S-O** key**.**

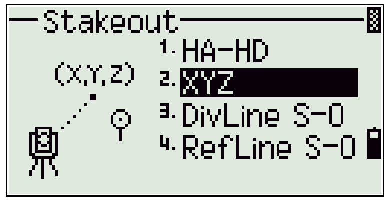

Fig. 10.5: Stakeout Menu Screen

- Select **2.XYZ** by pressing ② key**.**
- The **Stakeout Point Input Screen** is displayed.

Fig. 10.6: Stakeout Point Input Screen

- Input **Stakeout Point Number PT:** of first missing point to be "reset".
- The coordinates of the point to be staked should be displayed.
- Press **ENT** key to accept the point.
- The **Stakeout Observation Screen** showing **dHA** and **HD** is displayed.
    - **dHA:** is the zero countdown horizontal angle (turn in direction of arrow).
    - **HD:** is the horizontal distance to the stakeout point.

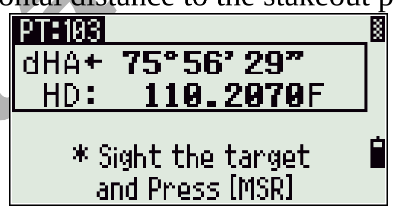

Fig. 10.7: Stakeout Observation Screen

- Rotate the instrument until **dHA** is **0°00'00"**. You are now "on-line".
- Sight the target position on-line and press **MSR1 or MSR2** key.
- The distance the prism is to be moved **In/Out** to be on the stakeout point is displayed.

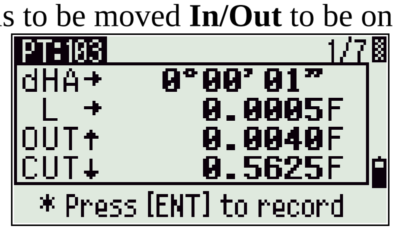

Fig. 10.8: In/Out Stakeout Screen

- Move the target and repeat measurements until **In/Out** distance is **0.000**.

10.1 Resetting Missing Points (cont'd)

4. Stakeout (cont'd)

- To record the coordinates of the target position
  - Press **REC** key to display the coordinates, point number and code.

Fig. 10.9: Coordinate Display Screen

-
  -
    - Change Point Number if necessary.
    - Enter Code if necessary.
  - Press **ENT** key to record the staked point.
- The **Stakeout Observation Screen** showing **dHA** and **HD** is displayed.
- Press **ESC** key to return to the **Stakeout Point Input Screen**.
- The **Stakeout Point Number PT:** of the next point to be staked is displayed.

Fig. 10.10: Stakeout Point Input Screen

- Change the **Stakeout Point Number PT:** to the next missing point to be "reset".
- Repeat until all missing points have been "reset" or "restaked".

11.0 Transferring an Elevation into a Job Site

11.1 Transferring an Elevation into a Job Site

This section assumes the user is familiar with the following sequence of procedures from previous chapters;

1. **Create a New Job or Open an Existing Job.**
2. **Upload or Manually Input required Data.**
3. **Instrument Setup.**
4. **Station Setup from a Known Point or an Unknown Point.**

   Note: It is not necessary to enter an elevation for the point over which the instrument is set as an elevation will be computed.
   Station setup is completed and the **Basic Measurement Screen** is displayed.

5. **Remote Benchmark (RBM)** to determine **Station Point Elevation.**
   - Press ⑦ **STN** key**.**

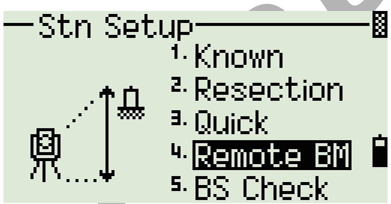

Fig. 11.1: Station Setup Menu Screen

- Select **4.Remote BM** by pressing ④ key.

Fig. 11.2: Benchmark Point Number Input Screen

- Input Benchmark Point Number **PT**, e.g. **36** and press **ENT**.
- The Benchmark Coordinates **N,E,Z** are displayed.

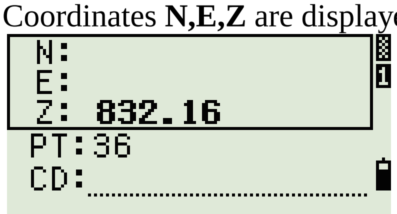

Fig. 11.3: Benchmark Coordinate Input Screen

- Input Benchmark Coordinates **N,E,Z** if displayed as blank.
  - Input Northing **N: (or leave blank)** and press **ENT** key.
  - Input Easting **E: (or leave blank)** and press **ENT** key.
  - Input Elevation **Z: (<u>Must</u>** enter a value).
- Input Code or description **CD (or leave blank)** of Benchmark and press **ENT**.

11.1 Transferring an Elevation into a Job Site (cont'd)

- Height of Target **HT Input Screen** is displayed.

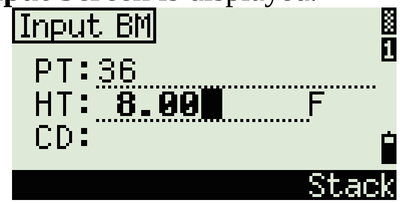

Fig. 11.4: Benchmark HT Input Screen

- Input **HT** and press **ENT** key.

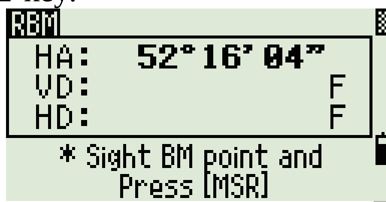

Fig. 11.5: Benchmark Sighting Screen

- Sight Benchmark Point and press **MSR1** key.
- Measurement data to Benchmark is displayed.

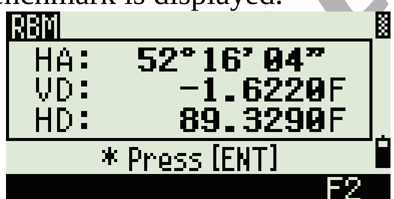

Fig. 11.6: Benchmark Measurement Data Screen

- Press **ENT** key.
- The updated station coordinates are displayed.

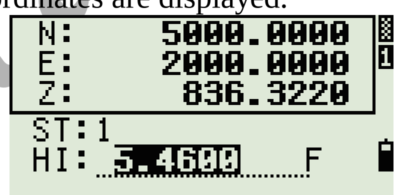

Fig. 11.7: Updated Station Coordinates Screen

  - Change Height of Instrument **HI** (if necessary) and press **ENT**.
- The updated station coordinates are stored.
  - **Note:** If the **HI** is changed, the **Z** coordinate is updated before the station coordinates are stored.
- **Basic Measurement Screen (BMS)** is displayed.

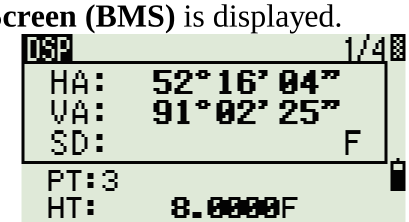

Fig. 11.8: Basic Measurement Screen

6. Data Collection or Stakeout.

Once the Remote Benchmark procedure is completed and the correct elevation for the instrument station is stored, the user may proceed to either **Data Collection or Stakeout.**

12.0 Blocking Diagram Layout

12.1 Layout from Blocking Diagrams

1. Instrument Setup.

- Set the instrument over the control or station point.
- Level the instrument.
- Power on the instrument.
- Tilt the telescope to initialize the Vertical Circle.
- **Basic Measurement Screen (BMS)** is displayed.

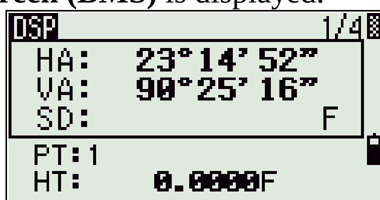

Fig. 12.1: Basic Measurement Screen

2. Create a Job

- Press **[Menu]** key**.**

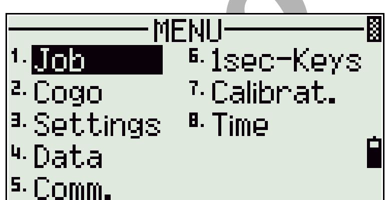

Fig. 12.2: Main Menu Screen

- Select **1.Job** by pressing ① key.

Fig. 12.3: Job Manager Screen

- Select **Creat** softkey by pressing **MSR1** key.

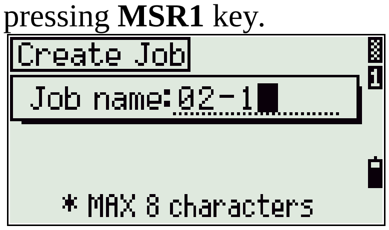

Fig. 12.4: Create Job Screen

- Input **Job Name** up to 8 characters.
- Press **ENT** key.

12.1 Layout from Blocking Diagrams (cont'd)

2. Create a Job (cont'd)

- **Create Job Confirmation Screen** is displayed.

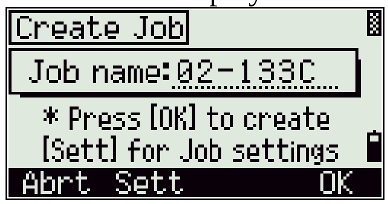

Fig. 12.5: Create Job Confirmation Screen

- Select **OK** softkey by pressing **ANG** key.
- **Basic Measurement Screen (BMS)** is displayed.

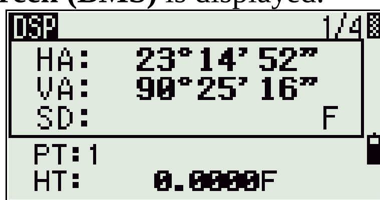

Fig. 12.6: Basic Measurement Screen

3. Station Setup

- Press ⑦ **STN** key**.**

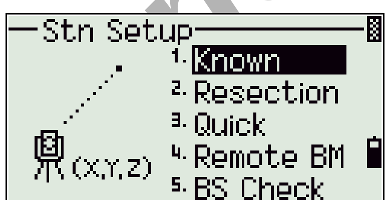

Fig. 12.7: Station Setup Menu Screen

- Select **1.Known** by pressing ① key.

Fig. 12.8: Station Input Screen

- Input Station Point Number **ST**, e.g. **1** and press **ENT**.

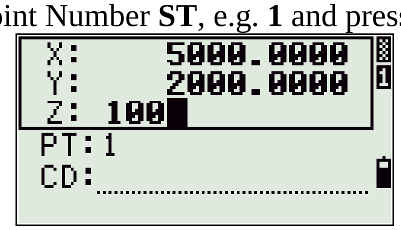

Fig. 12.9: Station Coordinates Input Screen

- Input Station Coordinates **N,E,Z** if displayed as blank and press **ENT**.
- Input Station Code or Description **CD** and press **ENT**.
- Input Station Height of Instrument **HI** and press **ENT**.

12.1 Layout from Blocking Diagrams (cont'd)

3. Station Setup (cont'd)

- **Backsight Menu Screen** is displayed.

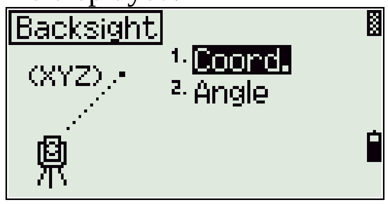

Fig. 12.10: Backsight Menu Screen

- Select either
  - **1.Coord.** if the backsight has known coordinates or
  - **2.Angle** if only the azimuth to the backsight is known.
- Perform backsight observation per prompt sequence below;

Fig. 12.11: Backsight Point Input Screen

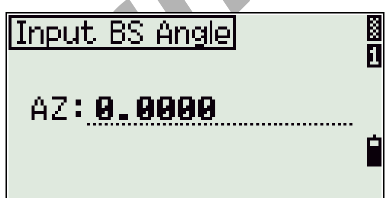

Fig. 12.12: Backsight Azimuth Input Screen

Fig. 12.13: Backsight Measurement Screen

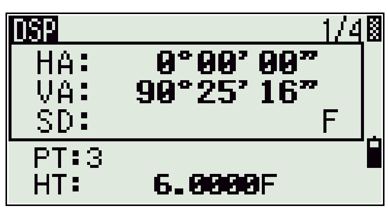

Fig. 12.14: Basic Measurement Screen

12.1 Layout from Blocking Diagrams (cont'd)

4. Measure & Record End Points on Baseline.

- Sight the first baseline endpoint **(P1)** to be collected.
- Press **MSR1** key.

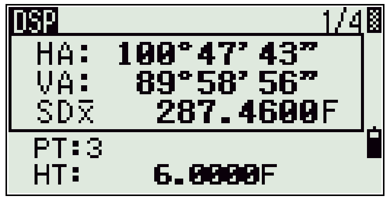

Fig. 12.15: Measurement Screen

- When distance **SD** is displayed, press **REC** key.
- A **"Record PT" Data Input Screen** is displayed.

Fig. 12.16: Record Point Input Screen

- Input Point Number **PT** if necessary.
- Input Height of Target **HT** if necessary.
- Input Code **CD** if necessary by manually typing, or
  - Select from a **List** by pressing **DSP** key, or
  - Select from a **Stack** by pressing **ANG** key.
- Press **ENT** key to record the point.

- Sight the second baseline endpoint **(P2)** to be collected.
- Press **MSR1** key.

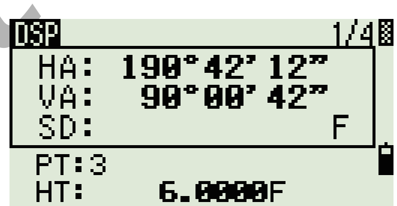

Fig. 12.17: Measurement Screen

- When distance **SD** is displayed, press **REC** key.
- A **"Record PT" Data Input Screen** is displayed.

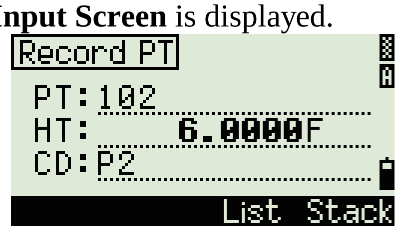

Fig. 12.18: Record Point Input Screen

- Input Point Number **PT** if necessary.
- Input Height of Target **HT** if necessary.
- Input Code **CD** if necessary by manually typing, or
  - Select from a **List** by pressing **DSP** key, or
  - Select from a **Stack** by pressing **ANG** key.
- Press **ENT** key to record the point.

12.1 Layout from Blocking Diagrams (cont'd)

5. Blocking Diagram Layout.

- Press ④ **PRG** key**.**

Fig. 12.19: PRG Menu Screen

- Select **1.2Pt RefLine** by pressing ① key**.**

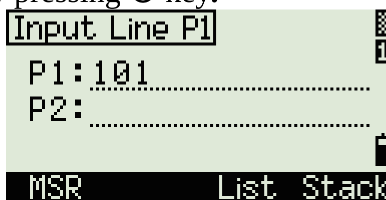

Fig. 12.20: Line Input Screen (P1)

- Input Point Number of First Baseline Point **P1:** and press **ENT** key.

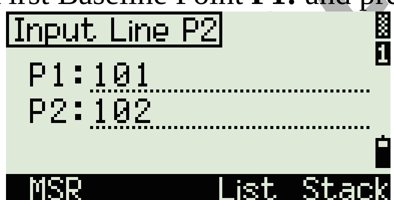

Fig. 12.21: Line Input Screen (P2)

- Input Point Number of Second Baseline Point **P2:** and press **ENT** key.
- The **Station & Offset Screen** showing **Sta** and **O/S** is displayed.

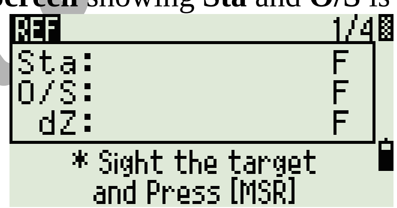

Fig. 12.22: Station & Offset Screen

  - **Sta:** is the horizontal distance along the baseline (P1-P2) from P1 to the point to be checked.
  - **O/S:** is the horizontal offset from the baseline (P1-P2) to the point to be checked.
- Sight the target position of the point to be checked and press **MSR1** key.
- The baseline **Station** and **Offset** values to the checked point are displayed.

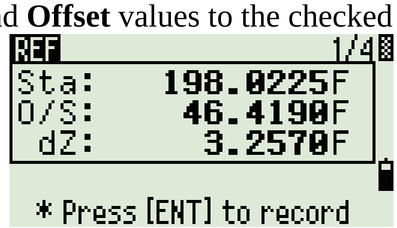

Fig. 12.23: Station & Offset Measurement Screen

12.1 Layout from Blocking Diagrams (cont'd)

5. Blocking Diagram Layout. (cont'd)

- Adjust the target position and repeat measurements until **Sta:** and **O/S** are correct.
- To record the location of the target position
    - Press **REC** key to display Point Number and Code.

Fig. 12.24: Record Point Input Screen

- Change Point Number if necessary.
- Enter Code if necessary.

- Press **ENT** key to record the checked point.
- Repeat until all points have been checked.

6. Download the Survey Data.

- Prepare computer to accept data.
- Press **[Menu]** key.

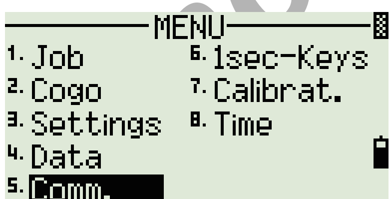

Fig. 12.25: Main Menu Screen

- Select **5.Comm.** by pressing ⑤.

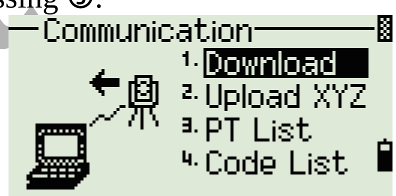

Fig. 12.26: Communications Menu Screen

- Select **1.Download** by pressing ①.
- **Download Settings Screen** is displayed.

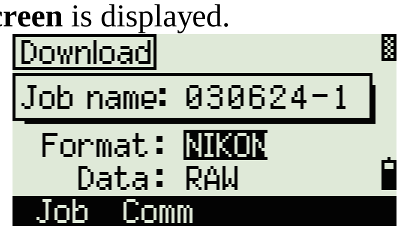

Fig. 12.27: Download Settings Screen

12.1 Layout from Blocking Diagrams (cont'd)

6. Download the Survey Data. (cont'd)

- Select
  - **Format: NIKON** and
  - **Data: RAW** and press **ENT** key.

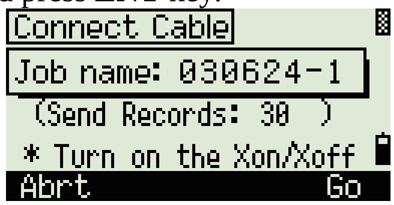

Fig. 12.28: Download Confirmation Screen

- Ensure cable is connected to instrument and computer.
- Select **Go** softkey by pressing **ANG** key.
- **SENDING screen** is displayed with record counter update till Complete.

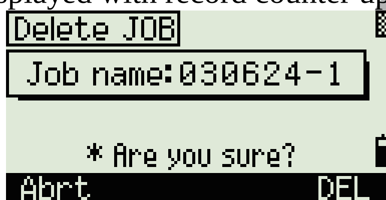

Fig. 12.29: Delete Job Screen

- At the **Delete JOB** screen, press either
  - **Abrt** softkey (**MSR1** key) to **NOT** Delete Job and return to **BMS Screen**, or
  - **DEL** softkey (**ANG** key) to Delete Job and return to **BMS Screen.**

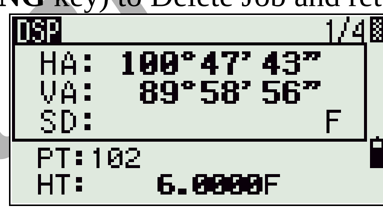

Fig. 12.30: Basic Measurement Screen

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GLOSSARY

Glossary of Surveying Terminology

Accuracy is the relationship between the value of a measurement and its "true or actual" value. The higher the accuracy, the smaller the errors in measurement.

Alidade is the upper assembly of the instrument which may be rotated about the base.

Angles are classified as Horizontal or Vertical according to the plane in which they are measured and are determined by the reference or starting line, the direction of turning and the angular distance or angular value.

Azimuths are angles measured clockwise from a reference meridian (generally North) and range from 0° to 360°.

Backsight is an initial reference point or direction from which angles and distances are measured to compute the positions of other points for stakeout and collection.

Bearings are a system of expressing directions of lines by means of an angle and quadrant letters. The bearing angle is the acute horizontal angle between the reference meridian and the line. The angle is measured from either the North or South to the East or West and always gives a reading less than or equal to 90°. **Examples:** N10°20'30"W, S18°E

Benchmark (BM) is a point of known elevation.

Centering is the precise alignment of the instrument's central axis over a point.

Circle reading is the current displayed value on the instrument for the Horizontal and/or Vertical angles.

COGO (CO̲ordinate G̲eO̲metry) refers to the mathematical computations performed in solving unknown relationships between points having known coordinates. Examples include calculation of length and direction of a line between two known points, the point(s) of intersection of two lines, a line and a distance or two distances, and area.

Coordinates define the position of a point by perpendicular distances from the axes of the coordinate system. Northing, Easting and Elevation are survey coordinates.

Coordinate Geometry (COGO) refers to the mathematical computations performed in solving unknown relationships between points having known coordinates. Examples include calculation of length and direction of a line between two known points, the point(s) of intersection of two lines, a line and a distance and two distances, and area.

Crosshairs are the horizontal and vertical lines in an instrument's telescope which are used to sight to the target.

Data collection is the process of taking survey field measurements and recording them internally in a total station or externally in a data recorder.

Direction of a line is the horizontal angle between the line and an arbitrarily chosen reference line termed a meridian.

Easting (E) coordinate of a point's position is the perpendicular distance from the point to the North-South Axis.

Elevation (Z) is the vertical distance above or below a reference (known or assumed) datum.

Free Station (Resection) is the determination of an instrument's position or coordinates by measurement to two or more points having known coordinates.

Height of Instrument (HI) is the height of the instrument above its station.

Height of Target (HT) is the height of the prism above its station.

Horizontal Angles (HA) are angles measured in the horizontal plane. May be the difference between two directions.

Horizontal Clamp is used to lock and prevent horizontal movement of the instrument.

Horizontal Distances (HD=SD cosθ) are distances computed in the horizontal plane representing the straight line distance between plumb lines at any two points.

Horizontal Tangent Screw is used to fine adjust the horizontal motion of the instrument.

Leveling is the precise vertical alignment of the instrument's vertical axis.

Meridian (Astronomic or true) is the North-South reference line through the earth's geographic poles.

Magnetic Meridian is the North-South reference line through the earth's magnetic poles.
Assumed Meridian is established by assigning any arbitrary direction to a line. Directions of all other lines are found relative to the assumed meridian.

Nadir point is a point on a vertical line directly beneath below the instrument position.

Northing coordinate of a point's position is the perpendicular distance from the point to the East-West Axis.

Online refers to the target being located at the correct horizontal angle usually in stakeout.

Optical Plummet is the smaller eyepiece near the base of the instrument which is used to ensure the instrument is centered over a point.

Plumb is a vertical line from the point to the center of the earth.

Prism refers to the reflective piece of glass used to reflect the light signal back to the total station.

Resection (Free Station) is the determination of an instrument's position or coordinates by measurement to two or more points having known coordinates.

Sighting refers to the aiming of the telescope at the target, bringing the target into focus and aligning the target with the center crosshairs of the reticle.

Slope Distances (SD) are distances measured along inclined planes representing the straight line distance from the center of the EDM to the center point of the reflecting prism.

Stakeout is the process of transferring points with known design positions to their locations in the field.

Station is a point on the ground, generally having known coordinates, above which the instrument is set.

Target is a point to which the instrument is being sighted for measurement purposes.

Telescope is the tube assembly on the instrument which contains the objective and eyepiece lenses.

Total Station is a fully electronic instrument with coaxial optics which measures horizontal angle, vertical angle and slope distance with a single pointing to the target.

Tribrach is the detachable base, containing the leveling screws, of the instrument.

Vernier Scale is a graduated precision scale used in conjunction with graduated horizontal and vertical circles to determine a more precise angular value.

Vertical Angles (VA) are angles measured in the vertical plane from the horizontal.

Vertical Clamp is used to lock and prevent vertical movement of the telescope.

Vertical Distances ($VD=SD \sin\theta$) are distances computed in the direction of gravity representing the vertical or elevation difference between the center of the EDM and the center point of the reflecting prism.

Vertical Tangent Screw is used to fine adjust the vertical motion of the telescope.

Zenith point is a point on a vertical line directly above the instrument position.

Zenith Angles are angles measured in the vertical plane from the Zenith.

UNIT IV GPS SURVEYING

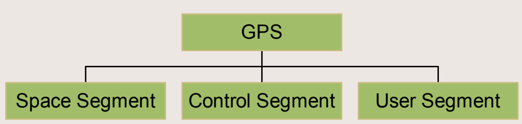
Seminar on GPS

Part I Working of GPS/DGPS
Part II Programming of GPS

Why do we need GPS?

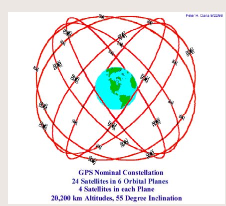
- Trying to figure out where you are is probable man's oldest pastime.
- Finally US Dept of Defense decided to form a worldwide positioning system.
- Also known as NAVSTAR (Navigation Satellite Timing and Ranging Global positioning system) provides instantaneous position, velocity and time information.

Components of the GPS



Space Segment:

- 24 GPS space vehicles(SVs).
- Satellites orbit the earth in 12 hrs.
- 6 orbital planes inclined at 55 degrees with the equator.
- This constellation provides 5 to 8 SVs from any point on the earth.



UNIT IV GPS SURVEYING

Control Segment:

Global Positioning System (GPS) Master Control and Monitor Station Network

- The control segment comprises of 5 stations.
- They measure the distances of the overhead satellites every 1.5 seconds and send the corrected data to Master control.
- Here the satellite orbit, clock performance and health of the satellite are determined and determines whether repositioning is required.
- This information is sent to the three uplink stations

User Segment:

- It consists of receivers that decode the signals from the satellites.
- The receiver performs following tasks:
  - Selecting one or more satellites
  - Acquiring GPS signals
  - Measuring and tracking
  - Recovering navigation data

User Segment:

- There are two services SPS and PPS
- <u>The Standard Positioning Service</u>
  - SPS- is position accuracy based on GPS measurements on single L1 frequency C/A code
  - C/A (coarse /acquisition or clear/access) GPs code sequence of 1023 pseudo random bi phase modulation on L1 freq

User Segment:

- <u>The Precise Position Service</u>
  - PPS is the highest level of dynamic positioning based on the dual freq P-code
  - The P-code is a very long pseudo-random bi phase modulation on the GPS carrier which does not repeat for 267 days
  - Only authorized users, this consists of SPS signal plus the P code on L1 and L2 and carrier phase measurement on L2

UNIT IV GPS SURVEYING

Cross Correlation

- Anti- spoofing denies the P code by mixing with a W-code to produce Y code which can be decoded only by user having a key.
- What about SPS users?
  - They use cross correlation which uses the fact that the y code are the same on both frequencies
  - By correlating the 2 incoming y codes on L1 and L2 the difference in time can be ascertained
  - This delay is added to L1 and results in the pseudorange which contain the same info as the actual P code on L2

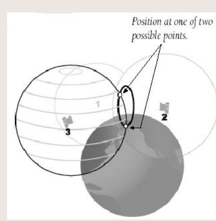
GPS Satellite Signal:

- L1 freq. (1575.42 Mhz) carries the SPS code and the navigation message.
- L2 freq. (1227.60 Mhz) used to measure ionosphere delays by PPS receivers
- 3 binary code shift L1 and/or L2 carrier phase
  - The C/A code
  - The P code
  - The Navigation message which is a 50 Hz signal consisting of GPs satellite orbits . Clock correction and other system parameters

How does the GPS work?

- Requirements
- Triangulation from satellite
- Distance measurement through travel time of radio signals
- Very accurate timing required
- To measure distance the location of the satellite should also be known
- Finally delays have to be corrected

Triangulation



- Position is calculated from distance measurement
- Mathematically we need four satellites but three are sufficient by rejecting the ridiculous answer

UNIT IV GPS SURVEYING

Measuring Distance

- Distance to a satellite is determined by measuring how long a radio signal takes to reach us from the satellite
- Assuming the satellite and receiver clocks are sync. The delay of the code in the receiver multiplied by the speed of light gives us the distance

Getting Perfect timing

- If the clocks are perfect sync the satellite range will intersect at a single point.
- But if imperfect the four satellite will not intersect at the same point.
- The receiver looks for a common correction that will make all the satellite intersect at the same point

Error Sources

- 95% due to hardware ,environment and atmosphere
- Intentional signal degradation
  - Selective availability
  - Anti spoofing

Selective Availabity

- Two components
  - Dither :
    manipulation of the satellite clock freq
  - Epsilon:
    errors imposed within the ephemeris data sent in the broadcast message

Anti spoofing

- Here the P code is made un gettable by converting it into the Y code.
- This problem is over come by cross correlation

Errors

- Satellite errors
  - Errors in modeling clock offset
  - Errors in Keplerian representation of ephemeris
  - Latency in tracking
- Atmospheric propagation errors
  - Through the ionosphere,carrier experiences phase advance and the code experiences group delay
  - Dependent on
    - Geomagnetic latitude
    - Time of the day
    - Elevation of the satellite

Errors

- Atmospheric errors can be removed by
  - Dual freq measurement

    low freq get refracted more than high freq

    thus by comparing delays of L1 and L2 errors can be eliminated
- Single freq users model the effects of the ionosphere

Errors

- Troposphere causes delays in code and carrier

  But they aren't freq dependent

  But the errors are successfully modeled
- Errors due to Multipath
- Receiver noise

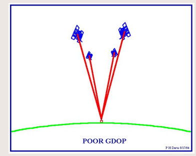
UNIT IV GPS SURVEYING

Errors

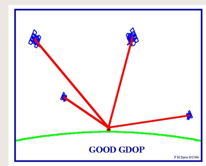
- Forces on the GPS satellite
  - Earth is not a perfect sphere and hence uneven gravitational potential distribution
  - Other heavenly bodies attract the satellite,but these are very well modeled
  - Not a perfect vacuum hence drag but it is negligible at GPS orbits
  - Solar radiation effects which depends on the surface reflectivity,luminosity of the sun,distance of to the sun. this error is the largest unknown errors source

Errors due to geometry

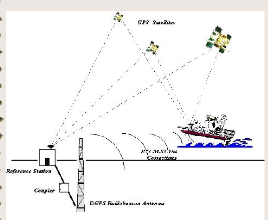
- Poor GDOP
  - When angles from the receiver to the SVs used are similar



- Good GDOP
  - When the angles are different



DGPS



- Errors in one position are similar to a local area
- High performance GPS receiver at a known location.
- Computes errors in the satellite info
- Transmit this info in RTCM-SC 104 format to the remote GPS

Requirements for a DGPS

- Reference station:
- Transmitter
  - Operates in the 300khz range
- DGPS correction receiver
  - Serial RTCM-SC 104 format
- GPS receiver

DGPS

- Data Links
  - Land Links
    - MF,LF,UHF/VHF freq used
    - Radiolocations,local FM, cellular telephones and marine radio beacons
  - Satellite links
    - DGPS corrections on the L band of geostaionary satellites
    - Corrections are determined from a network of reference Base stations which are monitored by control centers like OmniSTAR and skyFix

RTCM-SC 104 format

- DGPS operators must follow the RTCM-SC 104 format
- 64 messages in which 21 are defined
- Type 1 contains pseudo ranges and range corrections,issue of data ephemeris (IODE)and user differential range error(URDE)
- The IODE allows the mobile station to identify the satellite navigation used by the reference station.
- UDRE is the differential error determined by the mobile station

DGPS

- DGPS gives accuracy of 3-5 meters,while GPS gives accuracy of around 15-20 mts
- Removes the problem associated with SA.

Seminar On GPS

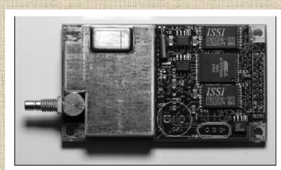


Figure 1. The Rockwell "Jupiter" GPS Receiver (-211) With Right Angle OSX (Top View -- Shown Approximately Twice Actual Size)

Part II

Programming Of GPS

(Rockwell "Jupiter" GPS Receiver)

Features:

- 12 parallel satellite tracking channels
- Supports NMEA-0183 data protocol & Binary data protocol.
- Direct, differential RTCM SC 104 data capability
- Static navigation improvements to minimize wander due to SA
- Active or Passive antenna to lower cost
- Max accuracy achievable by SPS
- Enhanced TTFF when in Keep –Alive power condition.
- Auto altitude hold mode from 3D to 2D navigation
- Maximum operational flexibility and configurable via user commands.
- Standard 2x10 I/O connector
- User selectable satellites

Satellite acquisition

- Jupiter GPS has 4 types of signal acquisition
  - Warm Start…..SRAM
  - Initialized start….EEPROM
  - Cold Start
  - Frozen Start

Table 2. "Jupiter" Receiver Signal Acquisition

	Time-To-First-Fix		Initial Error Uncertainties (3 Sigma)			Maximum Almanac Age	Maximum Ephemeris Age
Satellite Acquisition State	Typical (minutes)	90% Probable (minutes)	Position (km)	Velocity (m/sec)	Time (minutes)	Weeks	Hours
Warm	0.30	0.4	100	75	5	1	4
Initialized	0.8	1.0	100	75	5	1	N/A
Cold	2.0	2.5	N/A	N/A	N/A	1	N/A
Frozen	(*)	(*)	N/A	N/A	N/A	N/A	N/A

N/A = Not available in real-time to the receiver. Note that times are valid at 25 degrees Celsius with no satellite signal blockage.

(*) = Frozen start is considered to be a recovery mode. An "out-of-the-box" board that has not operated for a significant amount of time (months) may approximate this state because the data in EEPROM may be valid but expired or partially complete.

Navigation Modes

- 3D Navigation
  - At least 4 satellites
  - Computes latitude, longitude,altitude and time
- 2D Navigation
  - Less than 4 satellites or fixed altitude is given
- DGPS Navigation
  - Differential corrections are available through the auxiliary serial port
  - Must be in RTCM compliant

I/O interface of Jupiter

- Pins for powering GPS and Active antenna
- Two message formats NMEA and Binary
  - Pin 7 should be made high or low accordingly
- Two serial port
  - One is I/O….GPS data (Rx,Tx,Gnd)
  - Only input….RTCM format differential corrections (Rx,Gnd)
- Master reset pin(active low)
- Pin to provide battery backup

Selection o f mode

NMEA Protocol	ROM Default	Result
0	0	NMEA format, 4800bps 8N1
0	1	NMEA format, initial values from SRAM or EEPROM
1	0	Binary format,9600 8N1 From ROM
1	1	Data from SRAM or EEPROM

Serial data I/O interface

- Binary message format and NMEA format
- Binary message format
  - Header portion (compulsory)
  - Data portion (optional)

Binary message format

Header format

1000 0001 M L	1111 1111 M L
Message ID	
Data word count	
DCL0 QRAN	
Header checksum	

Binary Messages

- Example of binary messages:

  Aim: To disable the pinning feature

  Status of pinning is seen in User setting Output(Msg ID 1012) O/P message

  Pinning is controlled using Nav configuration (Msg ID 1221) I/P message

UNIT IV GPS SURVEYING

Binary messages

- I/p to the GPS to see the status of pinning
- Header format 81 ff sync word
  03 f4 Msg ID
  00 00 data count
  48 00 query bit set
  32 0d check sum

In response to this the GPS outputs User settings output message. (least significant byte first)

ff81 f403 1000 0048 ---- ---- ---- ---- 0000 ---- ----

The 5^{th} bit in the 9^{th} word of the above msg gives the status of pinning

Binary message

- I/p message to change status of pinning
- In the header
  - Msg Id becomes 04 C5 (nav configuration)
- Here the message also includes a data portion.
  - 2^{nd} bit of the 7^{th} word in the data portion is set to 1 to disable the pinning
  - The header checksum and data check sum must be correct for the message to be valid.
- Whether pining is disabled can be checked by sending the previous msg again. Now

ff81 f403 1000 0048 ---- ---- ---- ---- 7800 ---- ----

NMEA messages

- These are standardized sentences used in context with the GPS
- Examples: O/P statements
  - GGA: GPS fix Data
  - GSA: GPS DOP and active satellite
  - GSV: GPS Satellite in view
  - RMC: recommended min GPS data
- I/P messages
  - IBIT Built In test command
  - ILOG log control
  - INIT Initialization
  - IPRO Proprietary protocol

NMEA messages

Sample Message

$GPRMC,185203,A,1907.8900,N,07533.5546,E,0.00,121.7,221101,13.8,E*55

$ Start of sentence
Type of sentence
UTC
Validity
Latitude & orientation
Longitude & orientation
Speed
Heading
Date
Magnetic variation and orientation
Checksum (followed by <CR> and <LF>)

Connections with the GPS

- The signals available at the serial pins of the GPS are TTL level.
- To read the GPS output on Hyper terminal, the TTL signal is converted into RS 232 using a Max 232 IC
- The input messages are sent to the GPS using a simple C code

Conclusion:

- Components of the GPS
- Working of the GPS
- Errors sources in GPS
- Working of the DGPS
- Features of the Rockwell Jupiter GPS
- Binary and NMEA format
- Programming of the GPS

Thank you

UNIT –V

ASTRONOMICAL SURVEYING

Celestial sphere - Astronomical terms and definitions - Motion of sun and stars - Apparent altitude and corrections - Celestial co-ordinate systems - Different time systems –Use of Nautical almanac - Star constellations - calculations for azimuth of a line.

Celestial Sphere.

The millions of stars that we see in the sky on a clear cloudless night are all at varying distances from us. Since we are concerned with their relative distance rather than their actual distance from the observer. It is exceedingly convenient to picture the stars as distributed over the surface of an imaginary spherical sky having its center at the position of the observer. This imaginary sphere on which the star appear to lie or to be studded is known as the celestial sphere. The radius of the celestial sphere may be of any value – from a few thousand metres to a few thousand kilometers. Since the stars are very distant from us, the center of the earth may be taken as the center of the celestial sphere.

Zenith, Nadir and Celestial Horizon.

The Zenith (Z) is the point on the upper portion of the celestial sphere marked by plumb line above the observer. It is thus the point on the celestial sphere immediately above the observer's station.

The Nadir (Z') is the point on the lower portion of the celestial sphere marked by the plum line below the observer. It is thus the point on the celestial sphere vertically below the observer's station.

Celestial Horizon. (True or Rational horizon or geocentric horizon): It is the great circle traced upon the celestial sphere by that plane which is perpendicular to the Zenith-Nadir line, and which passes through the center of the earth. (Great circle is a section of a sphere when the cutting plane passes through the center of the sphere).

Terrestrial Poles and Equator, Celestial Poles and Equator.

The terrestrial poles are the two points in which the earth's axis of rotation meets the earth's sphere. The terrestrial equator is the great circle of the earth, the plane of which is at right angles to the axis of rotation. The two poles are equidistant from it.

If the earth's axis of rotation is produced indefinitely, it will meet the celestial sphere in two points called the north and south celestial poles (P and P'). The celestial equator is the great circle of the celestial sphere in which it is intersected by the plane of terrestrial equator.

Sensible Horizon and Visible Horizon.

It is a circle in which a plane passing through the point of observation and tangential to the earth's surface (or perpendicular to the Zenith-Nadir line) intersects with celestial sphere. The line of sight of an accurately leveled telescope lies in this plane.

It is the circle of contract, with the earth, of the cone of visual rays passing through the point of observation. The circle of contact is a small circle of the earth and its radius depends on the altitude of the point of observation.

Vertical Circle, Observer's Meridian and Prime Vertical?

A vertical circle of the celestial sphere is great circle passing through the Zenith and Nadir. They all cut the celestial horizon at right angles.

The Meridian of any particular point is that circle which passes through the Zenith and Nadir of the point as well as through the poles. It is thus a vertical circle.

It is that particular vertical circle which is at right angles to the meridian, and which, therefore passes through the east and west points of the horizon.

Latitude (θ) and Co-latitude (c).

Latitude (θ): It is angular distance of any place on the earth's surface north or south of the equator, and is measured on the meridian of the place. It is marked + or – (or N or S) according as the place is north or south of the equator. The latitude may also be defined as the angle between the zenith and the celestial equator.

The Co-latitude of a place is the angular distance from the zenith to the pole. It is the complement of the latitude and equal to (90°-θ).

longitude (φ) and altitude (α).

The longitude of a place is the angle between a fixed reference meridian called the prime of first meridian and the meridian of the place. The prime meridian universally adopted is that of Greenwich. Te longitude of any place varies between 0° and 180°, and is reckoned as Φ° east or west of Greenwich.

The altitude of celestial or heavenly body (i.e, the sun or a star) is its angular distance above the horizon, measured on the vertical circle passing through the body.

Co-altitude or Zenith Distance (z) and azimuth (A).

It is the angular distance of heavenly body from the zenith. It is the complement or the altitude, i.e $z = (90° - \alpha)$.

The azimuth of a heavenly body is the angle between the observer's meridian and the vertical circle passing through the body.

Declination (δ) and Co-declination or Polar Distance (p).

The declination of a celestial body is angular distance from the plane of the equator, measured along the star's meridian generally called the declination circle, (i.e., great circle passing through the heavenly body and the celestial pole). Declination varies from 0° to 90°, and is marked + or – according as the body is north or south of the equator.

It is the angular distance of the heavenly body from the near pole. It is the complement of the declination. i.e., $p = 90° - \delta$.

Hour Circle, Hour Angle and Right ascension (R.A).

Hour circles are great circles passing though the north and south celestial poles. The declination circle of a heavenly body is thus its hour circle.

The hour angle of a heavenly body is the angle between the observer's meridian and the declination circle passing through the body. The hour angle is always measured westwards.

Right ascension (R.A): It is the equatorial angular distance measured eastward from the First Point of Aries to the hour circle through the heavenly body.

Equinoctial Points.

The points of the intersection of the ecliptic with the equator are called the equinoctial points. The declination of the sun is zero at the equinoctial points. The Vernal Equinox or the First point of Aries (Y) is the sun's declination changes from south to north, and marks the commencement of spring. It is a fixed point of the celestial sphere. The Autumnal Equinox or the First Point of Libra (Ω) is the point in which sun's declination changes from north to south, and marks the commencement of autumn. Both the equinoctial points are six months apart in time.

ecliptic and Solstices?

Ecliptic is the great circle of the heavens which the sun appears to describe on the celestial sphere with the earth as a centre in the course of a year. The plan of the ecliptic is inclined to the plan of the equator at an angle (called the obliquity) of about 23° 27', but is subjected to a diminution of about 5" in a century.

Solstices are the points at which the north and south declination of the sun is a maximum. The point C at which the north declination of the sun is maximum is called the summer solstice; while the point C at which south declination of the sun is maximum is know as the winter solstice. The case is just the reverse in the southern hemisphere.

North, South, East and West Direction.

The north and south points correspond to the projection of the north and south poles on the horizon. The meridian line is the line in which the observer's meridian plane meets horizon place, and the north and south points are the points on the extremities of it. The direction ZP (in plan on the plane of horizon) is the direction of north, while the direction PZ is the direction of south. The east-west line is the line in which the prime vertical meets the horizon, and east and west points are the extremities of it. Since the meridian place is perpendicular to both the equatorial plan as well as horizontal place, the intersections of the equator and horizon determine the east and west points.

spherical excess and spherical Triangle?

The spherical excess of a spherical triangle is the value by which the sum of three angles of the triangle exceeds 180°.

Thus, spherical excess E = (A + B + C - 180°)

A spherical triangle is that triangle which is formed upon the surface of the sphere by intersection of three arcs of great circles and the angles formed by the arcs at the vertices of the triangle are called the spherical angles of the triangle.

Properties of a spherical triangle.

The following are the properties of a spherical triangle:

1. Any angle is less than two right angles or π.
2. The sum of the three angles is less than six right angles or 3π and greater than two right angles or π.
3. The sum of any two sides is greater than the third.
4. If the sum of any two sides is equal to two right angles or π, the sum of the angles opposite them is equal to two right angles or π.
5. The smaller angle is opposite the smaller side, and vice versa.

formulae involved in Spherical Trigonometry?

The six quantities involved in a spherical triangle are three angles A, B and C and the three sides a, b and c. Out of these, if three quantities are known, the other three can very easily be computed by the use of the following formulae in spherical trigonometry:

1. Sine formula: $\frac{\sin a}{\sin A} = \frac{\sin b}{\sin B} = \frac{\sin c}{\sin C}$

2. Cosine formula: $\cos A = \frac{\cos a - \cos b \cos c}{\sin b \sin c}$

Or $\cos a = \cos b \cos c + \sin b \sin c \cos A$

Also, $\cos A = - \cos B \cos C + \sin B \sin C \cos a$

systems used for measuring time?

There are the following systems used for measuring time:

1. Sidereal Time
2. Solar Apparent Time
3. Mean Solar Time
4. Standard Time

terrestrial latitude and longitude.

In order to mark the position of a point on the earth's surface, it is necessary to use a system of co-ordinates. The terrestrial latitudes and longitudes are used for this purpose.

The terrestrial meridian is any great circle whose plane passes through the axis of the earth (i.e., through the north and south poles). Terrestrial equator is great circle whose plane is perpendicular to the earth's axis. The latitude θ of a place is the angle subtended at the centre of the earth north by the are of meridian intercepted between the place and the equator.

The latitude is north or positive when measured above the equator, and is south or negative when measured below the equator. The latitude of a point upon the equator is thus 0°, while at the North and South Poles, it is 90° N and 90° S latitude respectively. The co-latitude is the complement of the latitude, and is the distance between the point and pole measured along the meridian.

The longitude (φ) of a place is the angle made by its meridian plane with some fixed meridian plane arbitrarily chosen, and is measured by the arc of equator intercepted between these two meridians. The prime meridian universally adopted is that of Greenwich. The longitude of any place varies between 0° to 180°, and is reckoned as φ° east or west of Greenwich. All the points on meridian have the same longitude.

Spherical Triangle? & its properties.

A spherical triangle is that triangle which is formed upon the surface of the sphere by intersection of three arcs of great circles and the angles formed by the arcs at the vertices of the triangle are called the spherical angles of the triangle.

AB, BC and CA are the three arcs of great circles and intersect each other at A, B and C. It is usual to denote the angles by A, B and C and the sides respectively opposite to them, as a, b and c. The sides of spherical triangle are proportional to the angle subtended by them at the centre of the sphere and are, therefore, expressed in angular measure. Thus, by sin b we mean the sine of the angle subtended at the centre by the arc AC. A spherical angle is an angle between two great circles, and is defined by the plane angle between the tangents to the circles at their point of intersection. Thus, the spherical angle at A is measured by the plane angle A1AA2 between the tangents AA1 and AA2 to the great circles AB and AC.

Properties of a spherical triangle

The following are the properties of a spherical triangle:

1. Any angle is less than two right angles or π.
2. The sum of the three angles is less than six right angles or 3π and greater than two right angles or π.
3. The sum of any two sides is greater than the third.
4. If the sum of any two sides is equal to two right angles or π, the sum of the angles opposite them is equal to two right angles or π.
5. The smaller angle is opposite the smaller side, and vice versa.

Formulae in Spherical Trigonometry

The six quantities involved in a spherical triangle are three angles A, B and C and the three sides a, b and c. Out of these, if three quantities are known, the other three can very easily be computed by the use of the following formulae in spherical trigonometry:

1. since formula : $$\frac{\sin a}{\sin A} = \frac{\sin b}{\sin B} = \frac{\sin c}{\sin C}$$

2. Cosine formula : $$\cos A = \frac{\cos a - \cos b \cos c}{\sin b \sin c}$$

or $$\cos a = \cos b \cos c + \sin b \sin c \cos A$$

Also, $\cos A = -\cos B \cos C + \sin B \sin C \cos a$

The Spherical Excess

The spherical excess of a spherical triangle is the value by which the sum of three angles of the triangle exceeds 180°.

Thus, spherical excess $E = (A + B + C - 180°)$

Also, $$\tan^2 \frac{1}{2} E = \tan \frac{1}{2} s \tan \frac{1}{2}(s-a) \tan \frac{1}{2}(s-b) \tan \frac{1}{2}(s-c)$$

In geodetic work the spherical triangles on the earth's surface are comparatively small and the spherical excess seldom exceeds more than a few seconds of arc. The spherical excess, in such case, can be expressed by the approximate formula

$$E = \frac{\Delta}{R2 \sin 1''} \text{ seconds}$$

where R is the radius of the earth and Δ is the area of triangle expressed in the same linear units as R.

the relationship between co-ordinates?

1. **The Relation between Altitude of the Pole and Latitude of the Observer.**

In the sketch, H-H is the horizon plane and E-E is the equatorial plane. O is the centre of the earth. ZO is perpendicular to HH while OP is perpendicular to EE.

Now latitude of place = $\theta = \angle EOZ$

And altitude of pole = $\alpha = \angle HOP$

$$\angle EOP = 90° = \angle EOZ + \angle ZOP$$
$$= \theta + \angle ZOP \quad \text{.... (i)}$$
$$\angle HOZ = 90° = \angle HOP + \angle POZ$$
$$= \alpha + \angle POZ \quad \text{.... (ii)}$$

Equating the two, we get

$\theta + \angle ZOP = \alpha + \angle POZ$ or $\theta = \alpha$

Hence the altitude of the pole is always equal to the latitude of the observer.

2. The Relation between Latitude of Observer and the Declination and Altitude of a Point on the Meridian.

For star M1, EM1 = δ = declination.

SM1 = α = meridian altitude of star.

M1Z = z = meridian zenith distance of star.

EZ = θ = latitude of the observer.

Evidently, EZ = EM1 + M1Z

Or $\theta = \delta + z$ …. (1)

The above equation covers all cases. If the star is below the equator, negative sign should be given to δ. If the star is to the north of zenith, negative sign should be given to z.

If the star is north of the zenith but above the pole, as at M2, we have

ZP = Z M2 + M2 P

or $(90° - \theta) = (90° - \alpha) + p$, where p = polar distance = M2 P

or $\theta = \alpha - p$ …. (2)

Similarly, if the star is north of the zenith but below the pole, as at M3, we have

ZM3 = ZP + PM3

$(90° - \alpha) = (90° - \theta) + p$, where p = polar distance = M3 P

$\theta = \alpha + p$ …. (3)

The above relations form the basis for the usual observation for latitude.

3. The Relation between Right Ascension and Hour Angle.

Fig 1.22. shows the plan of the stellar sphere on the plane of the equator. M is the position of the star and SPM is its westerly hour angle. HM. Y is the position of the First Point of Aries and angle SPY is its westerly hour angle. YPM is the rit ascension of the star. Evidently, we have

 Hour angle of Equinox = Hour angle of star + R.A. of star.

Find the difference of longitude between two places A and B from their following longitudes :]

(1) Longitude of A = 40° W

Longitude of B = 73° W

(2) Long. Of A = 20° E

Long. Of B = 150° E

(3) Longitude of A = 20° W

Longitude of B = 50° W

Solution.

(1) The difference of longitude between A and B = 73° - 40° = **33°**

(2) The difference of longitude between A and B = 150° - 20° = **130°**

(3) The difference of longitude between A and B = 20° - (- 50°) = **70°**

(4) The difference of longitude between A and B = 40° - (- 150°) = **190°**

Since it is greater than 180°, it represents the obtuse angular difference. The acute angular difference of longitude between A and B, therefore, is equal to

360° - 190° = **170°.**

Calculate the distance in kilometers between two points A and B along the parallel of latitude, given that

(1) Lat. Of A, 28° 42' N : longitude of A, 31° 12' W

Lat. Of B, 28° 42' N : longitude of B, 47° 24' W

(2) Lat. Of A, 12° 36' S : longitude of A, 115° 6' W

Lat. Of B, 12° 36' S : longitude of B, 150° 24' E.

Solution.

The distance in nautical miles between A and B along the parallel of latitude = difference of longitude in minutes x cos latitude.

(1) Difference of longitude between A and B = 47° 24' – 31° 12' =

16° 12' = 972 minutes

∴ Distance = 972 cos 28° 42' = 851.72 nautical miles

= 851.72 x 1.852 = **1577.34 km.**

(2) Difference of longitude between A and B

= 360° - { 115° 6' – (- 150° 24') } = 94° 30' = 5670 min.

 Distance = 5670 cos 12° 36’ = 5533.45 nautical miles

= 5533.45 x 1.852 = **10,247.2 km.**

Find the shortest distance between two places A and B, given that the longitudes of A and B are 15° 0’ N and 12° 6’ N and their longitudes are 50° 12’ E and 54° 0’ E respectively. Find also the direction of B on the great circle route.

Radius of earth = 6370 km.

Solution.

The positions of A and B have been shown.

In the spherical triangle ABP,

B = AP = 90° - latitude of A

= 90° - 15° 0’ = 75°

A = BP = 90° - latitude of B

= 90° - 12° 6’ = 77° 54’

P = A P B = difference of longitude

= 54° 0’ – 50° 12’ = 3° 48’.

The shortest distance between two points is the distance along the great circle passing through the two points.

Knowing the two sides one angle, the third side AB (=p) can be easily computed by the cosine rule.

Thus cos P = $\dfrac{\cos p - \cos a \cos b}{\sin a \sin b}$

or cos p = cos P sin a sin b + cos a cos b

= cos 3° 48’ sin 77° 54’ sin 75° + cos 77° 54’ cos 75°

= 0.94236 + 0.05425 = 0.99661

 p = AB = 4° 40’ = 4°.7

Now, arc ≈ radius x central angle = $\dfrac{6370 x 4°.7 x \pi}{180°} = 522.54 km.$

Hence distance AB = **522.54 km.**

Direction of A from B :

The direction of A from B is the angle B, and the direction of B from A is the angle A. Angles A and B can be found by the tangent semi-sum and semi-difference formulae

Thus $\tan \frac{1}{2}(A + B) = \dfrac{\cos \frac{1}{2}(a - b)}{\cos \frac{1}{2}(a + b)} \cot \frac{1}{2} p$

And $\tan \frac{1}{2}(A - B) = \dfrac{\sin \frac{1}{2}(a - b)}{\sin \frac{1}{2}(a + b)} \cot \frac{1}{2} p$

Here $\dfrac{(a - b)}{2} = \dfrac{77°54' - 75°}{2} = \dfrac{2°54'}{2} = 1°27'$

$$\frac{(a + b)}{2} = \frac{77°54' + 75°}{2} = \frac{152°54'}{2} = 76°27'; \frac{p}{2} = \frac{3°48'}{2} = 1°54'$$

∴ $\tan \frac{1}{2}(A + B) = \dfrac{\cos 1°27'}{\cos 76°27'} \cot 1°54$

From which, $\dfrac{A + B}{2} = 89°35'$ …. (i)

and $\tan \frac{1}{2}(A - B) = \dfrac{\sin 1°27'}{\sin 76°27'} \cot 1°54'$

From which, $\dfrac{A - B}{2} = 38°6'$ …. (ii)

∴ Direction of B from A = angle A = 89°35' + 38°6' = 127°41' = **S 52°19' E**

∴ Direction of A from B = angle B = 89°35' + 38°6' = 51°29' = **N 51°29' W.**

Determine the hour angle and declination of a star from the following data :

(i)	**Altitude of the star**	**= 22° 36'**
(ii)	**Azimuth of the star**	**= 42° W**
(iii)	**Latitude of the place of observation**	**= 40° N.**

Solution.

Since the azimuth of the star is 42° W, the star is in the western hemi-sphere.

In the astronomical Δ PZM, we have

PZ = co-latitude = 90° - 40° = 50° ;

ZM = co-altitude = 90° - 22° 36° = 67 24' ;

angle A = 42°

Knowing the two sides and the included angle, the third side can be calculated from the cosine formula

Thus, cos PM = cos PZ . cos ZM + sin PZ. Sin ZM. cos A

= cos 50°. cos 67° 24' + sin 50°. sin 67° 24'. cos 42°

= 0.24702 + 0.52556 = 0.77258

 PM = 39° 25'

 Declination of the star = δ = 90° - PM = 90° - 39° 25' = **50° 35' N.**

Similarly, knowing all the three sides, the hour angle H can be calculated from Eq. 1.2

$$\cos H = \frac{\cos ZM - \cos PZ.\cos PM}{\sin PZ.\sin PM} = \frac{\cos 67°24' - \cos 50°.\cos 39°25'}{\sin 50°.\sin 39°25'}$$

$$= \quad = \frac{0.38430 - 0.49659}{0.48640} = -0.23086$$

 cos (180° - H) = 0.23086 180° - H = 76° 39'

H = 103° 21'.

astronomical parameters of the earth and the sun.

The Earth:

The Earth is considered approximately spherical in shape. But actually it is very approximately an oblate spheroid. Oblate spheroid is the figure formed by revolving an ellipse about its minor axis. The earth is flattened at poles – its diameter along the polar axis being lesser than its diameter at the equator. The equatorial radius a of the earth, according to Hayford's spheroid is 6378.388 km and the polar radius b of the earth is 6356.912 km. The ellipticity is expressed by the ratio $\frac{a-b}{a}$, which reduces to $\frac{1}{297}$. For the Survey of India; Everest's first constants were used as follows:

$a = 20{,}922{,}932$ ft and $b = 20{,}853{,}642$ ft, the ellilticity being $\frac{1}{311.04}$.

The earth revolves about its minor or shorter axis (i.e. polar axis), on an average, once in twenty-four hours, from West to East. If the earth is considered stationary, the whole celestial sphere along with its celestial bodies like the stars, sum, moon etc. appear to revolve round the earth from East to West. The axis of rotation of earth is known as the polar axis, and the points at which it intersects the surface of earth are termed the North and South Geographical or Terrestrial Poles. In addition to the motion of rotation about its own polar axis, the earth has a motion of rotation relative to the sun, in a plane inclined at an angle of 23° 27' to the plane of the equator. The time of a complete revolution round the sun is one year. The apparent path of the sun in the heavens is the result of both the diurnal and annual real motions of the earth.

The earth has been divided into certain zones depending upon the parallels of latitude of certain value above and below the equator. The zone between the parallels of latitude 23° 27 ½ ' N and 23° 27 ½ ' S is known as the torrid zone (see Fig. 1.12). This is the hottest portion of the earth's surface. The belt included between 23° 27 ½ ' N and 66° 32 ½ ' N of equator is called the north temperate zone. Similarly, the belt included between 23° 27 ½ ' S and 66° 32 ½ ' S is called south temperate zone. The belt between 66° 32 ½ ' N and the north pole is called the north frigid zone and the belt between 66° 32 ½ ' S and the south pole is called south frigid zone.

The sun:

The sun is at a distance of 93,005,000 miles from the earth. The distance is only about $\frac{1}{250,000}$ of that of the nearest star. The diameter of the sun is about 109 times the diameter of the earth, and subtends and angle of 31' 59" at the centre of the earth. The mass of the sun is about 332,000 times that of the earth. The temperature at the centre of the sun is computed to be about 20 million degrees.

The sun has twp apparent motions, one with respect to the earth from east to west, and the other with respect to the fixed stars in the celestial sphere. The former apparent path of the sun is in the plane which passes through the centre of the celestial sphere and intersects it in a great circle called the ecliptic. The apparent motion of the sun is along this great circle. The angle between the plane of equator and the ecliptic is called the Obliquity of Ecliptic, its value being 23° 27'. The obliquity of ecliptic changes with a mean annual diminution of 0'.47.

The points of the intersection of the ecliptic with the equator are called the equinoctial points, the declination of the sun being zero at these points. The Vernal Equinox or the First point of Aries (Y) is the point in which the sun's declination changes from south to north. Autumnal Equinox or the First point of Libra ($\underline{\Omega}$) is the point in which the sun's declination changes from north to south. The points at which sun's declinations are a maximum are called solstices. The point at which the north declination of sun is maximum is called the summer solstice, while the point at which the south declination of the sun is maximum is known as the winter solstice.

The earth moves eastward around the sun once in a year in a pat that is very nearly a huge circle with a radius of about 93 millions of miles. More accurately, the path is described as an ellipse, one focus of the ellipse being occupied by the sun.

Various measurements of time.

Due to the intimate relationship with hour angle, right ascension and longitude, the knowledge of measurement of time is most essential. The measurement of time is based upon the apparent motion of heavenly bodies caused by earth's rotation on its axis. Time is the interval which lapses, between any two instants. In the subsequent pages, we shall use the following abbreviations.

G.M.T.	… Greenwhich Mean Time	G.M.M.	… Greenwich Mean Midnight
G.A.T.	… Greenwich Apparent Time	L.A.N.	… Local Apparent Noon
G.S.T.	… Greenwich Sidereal Time	L.M.M.	… Local Mean Midnight
L.M.T.	… Local Mean Time	L.Std.T.	… Local Standard Time
L.A.T.	… Local Apparent Time	N.A.	… Nautical Almanac
L.S.T.	… Local Sidereal Time	S.A.	… Star Almanac
G.M.N.	… Greenwich Mean Noon		

The units of time.

There are the following systems used for measuring time :

1. Sidereal Time
2. Solar Apparent Time
2. Mean Solar Time
4. Standard Time

Sidereal Time:

Since the earth rotates on its axis from west to east, all heavenly bodies (i.e. the sun and the fixed stars) appear to revolve from east to west (i.e. in clock-wise direction) around the earth. Such motion of the heavenly bodies is known as apparent motion. We may consider the earth to turn on it axis with absolute regular speed. Due to this, the stars appear to complete one revolution round the celestial pole as centre in constant interval of time, and they cross the observer's meridian twice each day. For astronomical purposes the sidereal day is one of the principal units of time. The sidereal day is the interval of time between two successive upper transits of the first point of Aries (Y). It begins at the instant when the first point of Aries records 0h, 0m, 0s. At any other instant, the sidereal time will be the hour angle of Y reckoned westward from 0h to 24h. The sidereal day is divided into 24 hours, each hour subdivided into 60 minutes and each minute into 60 seconds. However, the position of the Vernal Equinox is not fixed. It has slow (and variable) westward motion caused by the precessional movement of the axis, the actual interval between two transits of the equinox differs about 0.01 second of time from the true time of one rotation.

Local Sidereal Time (L.S.T.):

The local sidereal time is the time interval which has elapsed since the transit of the first point of Aries over the meridian of the place. It is, therefore, a measure of the angle through which the earth has rotated since the equinox was on the meridian. The local sidereal time is, thus, equal to the right ascension of the observer's meridian.

Since the sidereal time is the hour angle of the first point of Aries, the hour angle of a star is the sidereal time that has elapsed since its transit. M1 is the position of a star having SPM1 (= H) as its hour angle measured westward and YPM1 is its right ascension (R.A.) measured eastward. SPY is the hour angle of Y and hence the local sidereal time.

Hence, we have SPM1 + M1PY = SPY

or **star's hour angle + star's right ascension = local sidereal time** … (1)

If this sum is greater than 24 hours, deduct 24 hours, while if it is negative add, 24 hours.

The star M2 is in the other position. Y PM2 is its Right Ascension (eastward) and ZPM2 is its hour angle (westward). Evidently,

ZPM2 (exterior) + YPM2 – 24h = SPY = L.S.T.

or **star's hour angle + star's right ascension – 24h = L.S.T**

This supports the preposition proved with reference to Fig. 1.30 (a). The relationship is true for all positions of the star.

When the star is on the meridian, its hour angle is zero. Hence equation 1 reduces to

Star's right ascension = local sidereal time at its transit.

A sidereal clock, therefore, records the right ascension of stars as they make their upper transits.

The hour angle and the right ascension are generally measured in time in preference to angular units. Since one complete rotation of celestial sphere through 360° occupies 24 hours, we have

24 hours = 360° ; 1 hour = 15°

The difference between the local sidereal times of two places is evidently equal to the difference in their longitudes.

Solar Apparent Time:

Since a man regulates his time with the recurrence of light and darkness due to rising and setting of the sun, the sidereal division of time is not suited to the needs of every day life, for the purposes of which the sun is the most convenient time measurer. A solar day is the interval of time that elapes between two successive lower transits of the sun's centers over the meridian of the place. The lower transit is chosen in order that the date may change at midnight. The solar time at any instant is the hour angle of the sun's centre reckoned westward from 0h to 24h. This is called the apparent solar time, and is the time indicated by a sun-dial. Unfortunately, the apparent solar day is not of constant length throughout the year but changes. Hence our modern clocks and chronometers cannot be used to give us the apparent solar time. The non-uniform length of the day is due to two reasons :

(1) The orbit of the earth round the sun is not circular but elliptical with sun at one of its foci. The distance of the earth from the sun is thus variable. In accordance with the law of gravitation, the apparent angular motion of the sun is not uniform – it moves faster when is nearer to the earth and slower when away. Due to this, the sun reaches the meridian sometimes earlier and sometimes later with the result that the days are of different lengths at different seasons.

(2) The apparent diurnal path of the sun lies in the ecliptic. Due to this, even though the eastward progress of the sun in the ecliptic were uniform, the time elapsing between the departure of a meridian from the sun and its return thereto would vary because of the obliquity of the ecliptic.

The sun changes its right ascension from 0h to 24h in one year, advancing eastward among the stars at the rate of about 1° a day. Due to this, the earth will have to turn nearly 361° about its axis to complete one solar day, which will consequently be about minutes longer than a sidereal day. Both the obliquity of the ecliptic and the sun's unequal motion cause a variable rate of increase of the sun's right ascension. If the rate of change of the sun's right ascension were uniform, the solar day would be of constant length throughout the year.

Mean Solar Time :

Since our modern clocks and chronometers cannot record the variable apparent solar time, a fictitious sun called the mean sun is imagined to move at a uniform rate along the equator. The motion of the mean sun is the average of that of the true sun in its right ascension. It is supposed to start from the vernal equinox at the same time as the true sun and to return the vernal equinox with the true sun. The mean solar day may be defied as the interval between successive transit of the mean sun. The mean solar day is the average of all the apparent solar days of the year. The mean sun has the constant rate of increase of right ascension which is the average rate of increase of the true sun's right ascension.

The local mean noon (L.M.N.) is the instant when the mean sun is on the meridian. The mean time at any other instant is given by the hour angle of the mean sun reckoned westward from 0 to 24 hours. For civil purposes, however, it is found more convenient to begin the day at midnight and complete it at the next midnight, dividing it into two periods of 12 hours each. Thus, the zero hour of the mean day is at the local mean midnight (L.M.M.). The local mean time (L.M.T.) is that reckoned from the local mean midnight. The difference between the local mean time between two places is evidently equal to the difference in the longitudes.

From Fig. 1.30 (a) if M1 is the position of the sun, we have

Local sidereal time = R.A. of the sun + hour angle of the sun … (1)

Similarly,

Local sidereal time = R.A. of the mean sun + hour angle of the mean sun … (2)

The hour angle of the sun is zero at its upper transit. Hence

Local sidereal time of apparent noon = R.A. of the sun … (3)

Local sidereal time of mean noon = R.A. of the mean sun … (4)

Again, since the our angle of the sun (true or mean) is zero at its upper transit while the solar time (apparent or mean) is zero as its lower transit, we have

The apparent solar time = the hour angle of the sun + 12h … (5)

The mean solar time = the hour angle of mean sun + 12h … (6)

Thus, if the hour angle of the mean sun is 15° (1 hour) the mean time is 12 + 1 = 13 hours, which is the same thing as 1 o’clock mean time in the afternoon; if the hour angle of the mean sun is 195° (13 hours), the mean time is 12 + 13 =25 hours, which is the same as 1 o’clock mean time after the midnight (i.e., next. Day).

The Equation of Time

The difference between the mean and the apparent solar time at any instant is known as the equation of time. Since the mean sun is entirely a fictitious body, there is no means to directly observe its progress. Formerly, the apparent time was determined by solar observations and was reduced to mean time by equation of time. Now-a-days, however, mean time is obtained more easily by first determining the sidereal time by steller observations and then converting it to mean time through the medium of wireless signals. Due to this reason it is more convenient to regard the equation of time as the correction that must be applied to mean time to obtain apparent time. The nautical almanac tabulates the value of the equation of time for every day in the year, in this sense (i.e. apparent – mean). Thus, we have

Equation of time = Apparent solar time – Mean solar time

The equation of time is positive when the apparent solar time is more than the mean solar time ; to get the apparent solar time, the equation of time should then be added to mean solar time. For example, at 0h G.M.T. on 15 October 1949, the equation of the time is + 13m 12s. This means that the apparent time at 0h mean time is 0h 13m 12s. In other words, the true sun is 13m 12s ahead of the mean sun. Similarly, the equation of time is negative when the apparent time is less than the mean time. For example, at 0h G.M.T. on 18 Jan., 1949, the equation of time is – 10m 47s. This means that the apparent time at 0h mean time will be 23h 49m 13s on January 17. In other words, the true sun is behind the mean sun at that time.

The value of the equation of time varies in magnitude throughout the year and its value is given in the Nautical Almanac at the instant of apparent midnight for the places on the meridian of Greenwich for each day of the year. For any other time it must be found by adding or subtracting the amount by which the equation has increased or diminished since midnight.

It is obvious that the equation of time is the value expressed in time, of the difference at any instant between the respective hour angles or right ascensions of the true and mean suns.

The amount of equation of the time and its variations are due to two reasons :

(1) obliquity of the ecliptic, and (2) elasticity of the orbit. We shall discuss both the effects separately and then combine them to get the equation of time.

Explain the conversion of local time to standard time and vice versa.

The difference between the standard time and the local mean time at a place is equal to the difference of longitudes between the place and the standard meridian.

If the meridian of the place is situated east of the standard meridian, the sun, while moving apparently from east to west, will transit the meridian of the place earlier than the standard meridian. Hence the local time will be greater than the standard time. Similarly, if the meridian of the place is to the west of the standard meridian, the sun will transit the standard meridian earlier than the meridian of the place and hence the local time will be lesser than the standard time. Thus, we have

L.M.T = Standard M.T ± Difference in the longitudes $\left(\frac{E}{W}\right)$

L.A.T = Standard A.T ± Difference in the longitudes $\left(\frac{E}{W}\right)$

L.S.T = Standard S.T ± Difference in the longitudes $\left(\frac{E}{W}\right)$

Use (+) sign if the meridian of place is to the east of the standard meridian, and (-) Sign if it to the west of the standard meridian.

If the local time is to be found from the given Greenwich time, we have

L.M.T = Standard M.T ± Difference in the longitudes $\left(\frac{E}{W}\right)$

The standard time meridian in India is 82° 30’ E. If the standard time at any instant is 20 hours 24 minutes 6 seconds, find the local mean time for two places having longtitudes (a) 20° E, (b) 20° W.

Solution:

(a) The longitude of the place = 20° E

Longtitude of the standard meridian = 82° 30'E

 Difference in the longitudes = 82° 30' – 20° = 62° 30', E. the place being to the west of the standard meridian.

Now 62° of longitude $= \frac{62}{15}$ h $= 4^h\, 8^m\, 0^s$

Now 30' of longitude $= \frac{30}{15}$ m $= 0^h\, 2^m\, 0^s$

Total $= 4^h\, 10^m\, 0^s$

Now L.M.T = Standard time – Difference in longitude (W)

$= 20^h\, 24^m\, 6^s$ - -

Photogrammetry
- Introduction
- Terrestial and aerial Photographs
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Hydrographic Surveying
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Photogrammetric surveying?

Photogram metric surveying or photogrammetry is the science and art of obtaining accurate measurements by use of photographs, for various purposes such as the construction of planimetric and topographic maps, classification of soils, interpretation of geology, acquisition of military intelligence and the preparation of composite pictures of the ground. The photographs are taken either from the air or from station on the ground. Terrestrial photogrammetry is that Brach of photogrammetry wherein photographs are taken from a fixed position on or near the ground. Aerial photogrammetry is that branch of photogrammetry wherein the photographs are taken by a camera mounted in an aircraft flying over the area. Mapping from aerial photographs is the best mapping procedures yet developed for large projects, and are invaluable for military intelligence. The major users of aerial mapping methods are the civilian and military mapping agencies of the Government.

The conception of using photographs for purposes of measurement appears to have originated with the experiments of Aime Laussedat of the Corps of the French Army, who in 1851 produced the first measuring camera. He developed the mathematical analysis of photographs as perspective projections, thereby increasing their application to topography. Aerial photography from balloons probably began about 1858. Almost concurrently (1858), but independently of Laussedat, Meydenbauer in Germany carried out the first experiments in making critical measurements of architectural details by the intersection method in the basis of two photographs of the building. The ground photography was perfected in Canada by Capt. Deville, then Surveyor General of Canada in 1888. In Germany, most of the progress on the theoretical side was due to Hauck.

In 1901, Pulfrich in Jena introduced the stereoscopic principle of measurement and designed the stereo comparator. The stereoaitograph was designed (1909) at the Zeiss workshops in Jena, and this opened a wide field of practical application. Scheimpflug, an Australian captain, developed the idea of double projector in 1898. He originated the theory of perspective transformation and incorporated its principles in the photoperspecto graph. He also gave the idea of radial triangulation. His work paved the way for the development of aerial surveying and aerial photogrammetry.

In 1875, Oscar Messter built the first aerial camera in Germany and J.W.Bagloy and A.Brock produced the first aerial cameras in U.S.A. In 1923, Bauersfeld designed the Zeiss stereoplanigraph. The optical industries of Germany, Switzerland, Italy and France, and later also those of the U.S.A and U.S.S.R. took up the manufacture and constant further development of the cameras and plotting instruments. In World War II, both the sides made extensive use of aerial photographs for their military operations. World War II gave rise to new developments of aerial photography techniques, such as the application of radio control to photoflight navigation, the new wide-angle lenses and devices to achieve true vertical photographs.

principles behind terrestrial photogrammetry.

The principle of terrestrial photogrammetry was improved upon and perfected by Capt. Deville, then Surveyor General of Canada in 1888. In terrestrial photogrammetry, photographs are taken with the camera supported on the ground. The photographs are taken by means of a photo theodolite which is a combination of a camera and a theodolite. Maps are then compiled from the photographs.

The principle underlying the method of terrestrial photogrammetry is exactly similar to that of plane table surveying, i.e. if the directions of same objects photographed from two extremities of measured base are known, their position can be located by the intersection of two rays to the same object. However, the difference between this and plane tabling is that more details are at once obtained from the photographs and their subsequent plotting etc. is done by the office while in plane tabling all the detailing is done in the field itself.

Thus in Fig , A and B are the two stations at the ends of base AB. The arrows indicate the directions of horizontal pointing (in plan) of the camera. For each pair of pictures taken from the two ends, the camera axis is kept parallel to each other. From economy and speed point of view, minimum number of photographs should be used to cover the whole area and to achieve this, it is essential to select the best positions of the camera stations. A thorough study of the area should be done from the existing maps, and a ground reconnaissance should be made. The selection of actual stations depends upon the size and ruggedness of the area to be surveyed. The camera should be directed downward rather than upward, and the stations should be at the higher points on the area.

The terrestrial photogrammetry can be divided into two branches:

(i) Plane-table photogrammetry.

(ii) Terrestrial stereo photogrammetry

The plane table photogrammetry consists essentially in taking a photograph of the area to be mapped from each of the two or three stations. The photograph perpendiculars may be oriented at any angle to the base, but usually from an acute angle with the latter. The main difficulty arises in the identifications of image points in a pair of photographs. In the case of homogeneous areas of sand or grass, identification becomes impossible. The principles of stereo photogrammetry, however, produced the remedy.

In terrestrial stereo photogrammetry, due to considerable improvement of accuracy obtained by the stereoscopic measurement of pairs of photographs, the camera base and the angles of intersection of the datum rays to the points to be measured can be considerably reduced since the camera axes at the two stations exhibit great similarity to each other. The image points which are parallactically displaced relative to each other in the two photographs are fused to a single spatial image by the stereoscopic measurement.

shore line survey?

The shore line surveys consist of:

(i) Determination or delineation of shore lines,

(ii) Location of shore details and prominent features to which soundings may be connected,

(iii) Determination of low and high water lines for average spring tides,

The determination or delineation of shore lines is done by traversing along the shore and taking offsets to the water edge by tape, or stadia or plane table. If the river is narrow, both the banks may be located by running a single line of traverse on one bank. For wide rivers, however, transverse may be run along both the banks. The traverse should be

Connected at convenient intervals to check the work. Thus, the Fig. two traverses XY and **X – Y--** along the two opposite shores may be checked by taking observations from A and B to the points C and D. When the instrument is at B, angles ABC and ABD can be measured. From the measured length of AB and the four angles, the length CD can be calculated. If this agrees with the measured length of CD, the work is checked. Sometimes, a triangulation net is run along a wide river. In sea shore survey, buoys anchored off the shore and light houses are used as reference points and are located by triangulation.

In the case of tidal water, it is necessary to locate the high and low water lines. The position of high water line may be determined roughly from shore deposits and marks on rocks. To determine the high water line accurately, the elevation of mean high water of ordinary spring tide is determined and the points are located on the shore at that elevation as in direct method of contouring. The low water line can also be determined similarly. However, since the limited time is available for the survey of low water line, it is usually located by interpolation from soundings.

Sounding and the methods employed in sounding.

The measurement of depth below the water surface is called sounding. This corresponds to the ordinary spirit leveling in land surveying where depths are measured below a horizontal line established by a level. Here, the horizontal line or the datum is the surface of water, the level of which continuously goes on changing with time. The object of making soundings is thus to determine the configuration of the sub aqueous source. As stated earlier, soundings are required for:

(i) Making nautical charts for navigation;

(ii) Measurement of areas subject to scour or silting and to ascertain the quantities of dredged material;

(iii) Making sub-aqueous investigations to secure information needed for the construction, development and improvement of port facilities.

For most of the engineering works, soundings are taken form a small boat. The equipment needed for soundings are:

(i) Sounding boat (ii) Sounding rods or poles

(iii) Lead lines (iv) Sounding machine

(v) Fathometer.

Sounding boat

A row-boat for sounding should be sufficiently roomy and stable. For quiet water, a flat bottom boat is more suitable, but for rough water round-bottomed boat is more suitable. For regular soundings, a row boat may be provided with a well through which sounds are taken. A sounding platform should be built for use in smaller boat. It should be extended far enough over the side to prevent the line from striking the boat. If the currents are strong, a motor or stream launch may be used with advantage.

Sounding rods or poles

A sounding rod is a pole of a sound straight-grained well seasoned tough timber usually 5 to 8 cm in diameter and 5 to 8 metres long. They are suitable for shallow and quiet waters. An arrow or lead shoe of sufficient weights fitted at the end. This helps in holding them upright in water. The lead or weight should be of sufficient area so that it may not sink in mud or sand. Between soundings it is turned end for end without removing it from the water. A pole of 6 m can be used to depths unto 4 meters.

Lead lines

A lead line or a sounding line is usually a length of a cord, or tiller rope of Indian hemp or braided flax or a brass chain with a sounding lead attached to the end. Due to prolonged use, a line of hemp or cotton is liable to get stretched. To graduate such a line, it is necessary to stretch it thoroughly when wet before it is graduated. The line should be kept dry when not in use. It should be soaked in water for about one hour before it is used for taking soundings. The length of the line should be tested frequently with a tape. For regular sounding, a chain of brass, steel or iron is preferred. Lead lines are usually used for depths over about 6 meters.

Sounding lead is a weight (made of lead) attached to the line. The weight is conical in shape and varies from 4 to 12 kg depending upon the depth of water and the strength of the current. The weight should be somewhat streamlined and should have an eye at the top for attaching the cord. It often has cup-shaped cavity at the bottom so that it may be armed with land or tallow to pick up samples from the bottom. Where the bottom surface is soft, leadfilled pipe with a board at the top is used with the lead weight. The weight penetrates in the mud and stops where the board strikes the mud surface.

Suggested system of marking poles and lead lines

The U.S. Coast and Geodetic survey recommends the following system of marking the poles and the lead lines :

Poles : Make a small permanent notch at each half foot. Paint the entire pole white and the spaces between the 2- and 3-, the 7- and 8-and the 12- and 13-ft marks black. Point ½'' red bands at the 5- and 10-ft marks, a ½'' in black band at each of the other foot marks and ¼'' bands at the half foot marks. These bands are black where the pole is white and vice versa.

Lead Lines : A lead line is marked in feet as follow :

Feet	Marks
2, 12, 22 etc	Red bunting
4, 14, 24 etc	White bunting
6, 16, 26 etc	Blue bunting
8, 18, 28 etc	Yellow bunting
10, 60, 110 etc	One strip of leather
20, 70, 120 etc	Two strips of leather
30, 80, 130 etc	Leather with two holes
40, 90, 140 etc	Leather with one holes
50	Star-shaped leather
100	Star-shaped leather with one hole

The intermediate odd feet (1,3,5,7,9 etc.) are marked by white seizing.

Sounding Machine

Where much of sounding is to done, a sounding machine as very useful. The sounding machine may either be hand driven or automatic. Fig.4.3. show a typical hand driven Weddele's sounding machine.

The lead weight is carried at the end of a flexible wire cord attached to the barrel and can lowered at any desired rate, the speed of the drum being controlled by means of a break.

The readings are indicated in two dials—the outer dial showing the depth in feet and the inner showing tenths of a foot. A handle is used to raise the level which can be suspended at any height by means of a paul and ratchet. The sounding machine is mounted in a sounding boat and can be used up to a maximum depth of 100 ft.

Fathometer : Echo-sounding

A Fathometer is used in ocean sounding where the depth of water is too much, and to make a continuous and accurate record of the depth of water below the boat or ship at which it is installed. It is an *echo-sounding* instrument in which water depths are obtained be determining the time required for the sound waves to travel from a point near the surface of the water to the bottom and back. It is adjusted to read depth on accordance with the velocity of sound in the type of water in which it is being used. A fathometer may indicate the depth visually or indicate graphically on a roll which continuously goes on revolving and provide a virtual profile of the lake or sea.

What are the components of echo sounding instrument? Briefly explain the advantages of echo sounding.

The main parts of an echo-sounding apparatus are:

1. Transmitting and receiving oscillators.
2. Recorder unit.
3. Transmitter / Power unit.

Figure illustrates the principal of echo-sounding. It consists in recording the interval of time between the emission of a sound impulse direct to the bottom of the sea and the reception of the wave or echo, reflected from the bottom. If the speed of sound in that water is v and the time interval between the transmitter and receiver is t, the depth h is given by

$$h = \tfrac{1}{2} vt \qquad \ldots$$

Due to the small distance between the receiver and the transmitter, a slight correction is necessary in shallow waters. The error between the true depth and the recorded depth can be calculated very easily by simple geometry. If the error is plotted against the recorded depth, the true depth can be easily known. The recording of the sounding is produced by the action of a small current passing through chemically impregnated paper from a rotating stylus to an anode plate. The stylus is fixed at one end of a radial arm which revolves at constant speed. The stylus makes a record on the paper at the instants when the sound impulse is transmitted and when the echo returns to the receiver.

Advantage of echo-sounding

Echo-sounding has the following advantages over the older method of lead line and rod:

1. It is more accurate as a truly vertical sounding is obtained. The speed of the vessel does deviate it appreciably from the vertical. Under normal water conditions, in ports and harbors an accuracy of 7.5 cm may be obtained.

2. It can be used when a strong current is running and when the weather is unsuitable for the soundings to be taken with the lead line.

3. It is more sensitive than the lead line.

4. A record of the depth is plotted immediately and provides a continuous record of the bottom as the vessel moves forward.

5. The speed of sounding and plotting is increased.

6. The error due to estimation of water level in a choppy sea is reduced owing to the instability of the boat.

7. Rock underlying softer material is recorded and this valuable information is obtained more cheaply than would be the case where sub-marine borings are taken.

Making the soundings

If the depth is less than 25 m, the soundings can be taken when the boat is in motion. In the case of soundings with rod the leadsman stands in the bow and plunges the rod at a forward angle, depending on the speed o the boat, such that the rod is vertical when the boat reaches the point at which soundings is being recorded. The rod should be read very quickly. The nature of the bottom should also be recorded at intervals in the note-book.

If the sounding is taken with a lead, the leadsman stands in the bow of the boat and casts the lead forward at such a distances that the line will become vertical and will reach the bottom at a point where sounding is required. The lead is withdrawn from the water after the reading is taken. If the depth is great, the lead is not withdrawn from the water, but is lifted between the soundings.

The water surface, which is also the reference datum, changes continuously. It is, therefore, essential to take the readings of the tide gauges at regular interval so that the soundings can be reduced to a fixed datum. To co-relate each sounding with the gauge reading, it is essential to record the time at which each sounding is made.

What are the methods employed in locating soundings?

The soundings are located with reference to the shore traverse by observations made (i) entirely from the boat, (ii) entirely from the shore or (iii) from both.

The following are the methods of location

1. By cross rope.
2. By range and time intervals.
3. By range and one angle from the shore.
4. By range and one angle from the boat.
5. By two angles from the shore.
6. By two angles from the boat.
7. By one angle from shore and one from boat.
8. By intersecting ranges.
9. By tacheometry.

Range.

A range or range line is the line on which soundings are taken. They are, in general, laid perpendicular to the shore line and parallel to each other if the shore is straight or are arranged radiating from a prominent object when the shore line is very irregular.

Shore signals.

Each range line is marked by means of signals erected at two points on it at a considerable distance apart. Signals can be constructed in a variety of ways. They should be readily seen and easily distinguished from each other. The most satisfactory and economic type of signal is a wooden tripod structure dressed with white and coloured signal of cloth. The position of the signals should be located very accurately since all the soundings are to be located with reference to these signals.

Location by Cross-Rope

This is the most accurate method of locating the soundings and may be used for rivers, narrow lakes and for harbours. It is also used to determine the quantity of materials removed by dredging the soundings being taken before and after the dredging work is done. A single wire or rope is stretched across the channel etc. as shown in Fig.4.6 and is marked by metal tags at appropriate known distance along the wire from a reference point or zero station on shore. The soundings are then taken by a weighted pole. The position of the pole during a sounding is given by the graduated rope or line.

In another method, specially used for harbours etc., a reel boat is used to stretch the rope. The zero end of the rope is attached to a spike or any other attachment on one shore. The rope is would on a drum on the reel boat. The reel boat is then rowed across the line of sounding, thus unwinding the rope as it proceeds. When the reel boat reaches the other shore, its anchor is taken ashore and the rope is wound as tightly as possible. If anchoring is not possible, the reel is taken ashore and spiked down. Another boat, known as the sounding boat, then starts from the previous shore and soundings are taken against each tag of the rope. At the end of the soundings along that line, the reel boat is rowed back along the line thus winding in the rope. The work thus proceeds.

Location by Range and Time Intervals

In this method, the boat is kept in range with the two signals on the shore and is rowed along it at constant speed. Soundings are taken at different time intervals. Knowing the constant speed and the total time elapsed at the instant of sounding, the distance of the total point can be known along the range. The method is used when the width of channel is small and when great degree of accuracy is not required. However, the method is used in conjunction with other methods, in which case the first and the last soundings along a range are located by angles from the shore and the intermediate soundings are located by interpolation according to time intervals.

Location by Range and One Angle from the Shore

In this method, the boat is ranged in line with the two shore signals and rowed along the ranges. The point where sounding is taken is fixed on the range by observation of the angle from the shore. As the boat proceeds along the shore, other soundings are also fixed by the observations of angles from the shore. Thus B is the instrument station, **A1 A2** is the range along which the boat is rowed and **α1, α2, α3** etc., are the angles measured at B from points 1, 2, 3 etc. The method is very accurate and very convenient for plotting. However, if the angle at the sounding point (say angle β) is less than 30°, the fix becomes poor. The nearer the intersection angle (β) is to a right angle, the better. If the angle diminishes to about 30° a new instrument station must be chosen. The only defect of the method is that the surveyor does not have an immediate control in all the observation. If all the points are to be fixed by angular observations from the shore, a note-keeper will also be required along with the instrument man at shore since the observations and the recordings are to be done rapidly. Generally, the first and last soundings and every tenth sounding are fixed by angular observations and the intermediate points are fixed by time intervals. Thus the points with round mark are fixed by angular observations from the shore and the points with cross marks are fixed by time intervals. The arrows show the course of the boat, seaward and shoreward on alternate sections.

To fix a point by observations from the shore, the instrument man at B orients his line of sight towards a shore signal or any other prominent point (known on the plan) when the reading is zero. He then directs the telescope towards the leadsman or the bow of the boat, and is kept continually pointing towards the boat as it moves. The surveyor on the boat holds a flag for a few seconds and on the fall of the flag, the sounding and the angle are observed simultaneously.

The angles are generally observed to the nearest 5 minutes. The time at which the flag falls is also recorded both by the instrument man as well as on the boat. In order to avoid acute intersections, the lines of soundings are previously drawn on the plan and suitable instrument stations are selected.

Location by Range and One Angle from the Boat

The method is exactly similar to the previous one except that the angular fix is made by angular observation from the boat. The boat is kept in range with the two shore signals and is rowed along it. At the instant the sounding is taken, the angle, subtended at the point between the range and some prominent point B on the sore is measured with the help of sextant. The telescope is directed on the range signals, and the side object is brought into coincidence at the instant the sounding is taken. The accuracy and ease of plotting is the same as obtained in the previous method. Generally, the first and the last soundings, and some of the intermediate soundings are located by angular observations and the rest of the soundings are located by time intervals.

As compared to the previous methods, this method has the following advantages :

1. Since all the observations are taken from the boat, the surveyor has better control over the operations.

2. The mistakes in booking are reduced since the recorder books the readings directly as they are measured.

3. On important fixes, check may be obtained by measuring a second angle towards some other signal on the shore.

4. The obtain good intersections throughout, different shore objects may be used for reference to measure the angles.

Location by Two Angles from the Shore

In this method, a point is fixed independent of the range by angular observations from two points on the shore. The method is generally used to locate some isolated points. If this method is used on an extensive survey, the boat should be run on a series of approximate ranges. Two instruments and two instrument men are required. The position of instrument is selected in such a way that a strong fix is obtained. New instrument stations should be chosen when the intersection angle (θ) falls below 30°. Thus A and B are the two instrument stations. The distance d between them is very accuarately measured. The instrument stations A and B are precisely connected to the ground traverse or triangulation, and their positions on plan are known. With both the plates clamped to zero, the instrument man at A bisects B ; similarly with both the plates clamped to zero, the instrument man at B bisects A. Both the instrument men then direct the line of sight of the telescope towards the leadsman and continuously follow it as the boat moves. The surveyor on the boat holds a flag for a few seconds, and on the fall of the flag the sounding and the angles are observed simultaneously. The co-ordinates of the position P of the sounding may be computed from the relations :

The method has got the following advantages:

1. The preliminary work of setting out and erecting range signals is eliminated.

2. It is useful when there are strong currents due to which it is difficult to row the boat along the range line.

The method is, however, laborious and requires two instruments and two instrument men.

Location by Two Angles from the Boat

In this method, the position of the boat can be located by the solution of the three-point problem by observing the two angles subtended at the boat by three suitable shore objects of known position. The three-shore points should be well-defined and clearly visible. Prominent natural objects such as church spire, lighthouse, flagstaff, buoys etc., are selected for this purpose. If such points are not available, range poles or shore signals may be taken. Thus A, B and C are the shore objects and P is the position of the boat from which the angles α and β are measured. Both the angles should be observed simultaneously with the help of two sextants, at the instant the sounding is taken. If both the angles are observed by surveyor alone, very little time should be lost in taking the observation. The angles on the circle are read afterwards. The method is used to take the soundings at isolated points. The surveyor has better control on the operations since the survey party is concentrated in one boat. If sufficient number of prominent points are available on the shore, preliminary work o setting out and erecting range signals is eliminated. The position of the boat is located by the solution of the three point problem either analytically or graphically.

Location by One Angle from the Shore and the other from the Boat

This method is the combination of methods 5 and 6 described above and is used to locate the isolated points where soundings are taken. Two points A and B are chosen on the shore, one of the points (say A) is the instrument station where a theodolite is set up, and the other (say B) is a shore signal or any other prominent object. At the instant the sounding is taken at P, the angle α at A is measured with the help of a sextant. Knowing the distance d between the two points A and B by ground survey, the position of P can be located by calculating the two co-ordinates x and y.

Location by Intersecting Ranges

This method is used when it is required to determine by periodical sounding at the same points, the rate at which silting or scouring is taking place. This is very essential on the harbors and reservoirs. The position of sounding is located by the intersection of two ranges, thus completely avoiding the angular observations. Suitable signals are erected at the shore. The boat is rowed along a range perpendicular to the shore and soundings are taken at the points in which inclined ranges intersect the range, as illustrated in Fig. 4.12. However, in order to avoid the confusion, a definite system of flagging the range poles is necessary. The position of the range poles is determined very accurately by ground survey.

Location by Tacheometric Observations

The method is very much useful in smooth waters. The position of the boat is located by tacheometric observations from the shore on a staff kept vertically on the boat. Observing the staff intercept s at the instant the sounding is taken, the horizontal distance between the instrument stations and the boat is calculated by

The direction of the boat (P) is established by observing the angle (α) at the instrument station B with reference to any prominent object A The transit station should be near the water level so that there will be no need to read vertical angles. The method is unsuitable when soundings are taken far from shore.

Explain reduction of soundings with a example.

The reduced soundings are the reduced levels of the sub-marine surface in terms of the adopted datum. When the soundings are taken, the depth of water is measured with reference to the existing water level at that time. If the gauge readings are also taken at the same time, the soundings can be reduced to a common unvarying datum. The datum most commonly adopted is the 'mean level of low water of spring tides' and is written either as L.W.O.S.T. (low water, ordinary spring tides) or

M.L.W.S. (mean low water springs). For reducing the soundings, a correction equal to the difference of level between the actual water level (read by gauges) and the datum is applied to the observed soundings, as illustrated in the table given below :

Gauge Reading at L.W.O.S.T. = 3.0 m.

Time	Gauge (m)	Distance	Soluction (m)	Correction	Reduced sounding (m)	Remarks
8.00 A.M.	3.5	10	2.5	-0.5	2.00	
		20	3.2		2.7	
		30	3.9		3.4	
		40	4.6		4.1	
8.10 A.M.	3.5	50	5.3	-0.5	4.8	
		60	5.4		4.9	
		70	5.1		4.6	
		80	4.7		4.2	
		90	3.6		3.1	
8.10 A.M.	3.5	100	2.1	-0.5	1.6	

What is three point problem ?How it can be solved ?

Given the three shore signals A, B and C, and the angles α and β subtended by AP, BP and CP at the boat P, it is required to plot the position of P

1. Mechanical Solution

(i) By Tracing Paper

Protract angles α and β between three radiating lines from any point on a piece of tracing paper. Plot the positions of signals A, B, C on the plan. Applying the tracing paper to the plan, move it about until all the three rays simultaneously pass through A, B and C. The apex of the angles is then the position of P which can be pricked through.

(ii) By Station Pointer :

The station pointer is a three-armed protractor and consists of a graduated circle with fixed arm and two movable arms to the either side of the fixed arm. All the three arms have beveled or fiducial edges. The fiducial edge of the central fixed arm corresponds to the zero of the circle. The fiducial edges of the two moving arms can be set to any desired reading and can be clamped in position. They are also provided with verniers and slow motion screws to set the angle very precisely. To plot position of P, the movable arms are clamped to read the angles α and β very precisely. The station pointer is then moved on the plan till the three fiducial edges simultaneously touch A, B and C. The centre of the pointer then represents the position of P which can be recorded by a prick mark.

2. Graphical Solutions

(a) First Method :

Let a, b and c be the plotted positions of the shore signals A, B and C respectively and let α and β be the angles subtended at the boat. The point p of the boat position p can be obtained as under :

1. Join a and c.
2. At a, draw ad making an angle β with ac. At c, draw cd making an angle α with ca. Let both these lines meet at d.
3. Draw a circle passing through the points a, d and c.
4. Join d and b, and prolong it to meet the circle at the point p which is the required position of the boat.

Proof. From the properties of a circle,

$\angle$ apd = $\angle$ acd = α and $\angle$ cpd = $\angle$ cad = β

which is the required condition for the solution.

(b) Second Method :

Join ab and bc.

1. From a and b, draw lines **ao1 and bo1** each making an angle (90° - α) with ab on the side towards p. Let them intersect at **01.**
2. Similarly, from b and c, draw lines ================ each making an angle (90° - β) with ab on the side towards p. Let them intersect at --.
3. With – as the centre, draw a circle to pass through a and b. Similarly, with – as the centre draw a circle to pass through b and c. Let both the circles intersect each other at a point p. p is then the required position of the boat.

Proof.	$\angle$ ao1b	= 180° - 2 (90° - α) = 2α
	$\angle$ apb	= ½ $\angle$ ao1b = α
Similarly,	$\angle$ bo2c	= 180° - 2 (90° - β) = 2β
and	$\angle$ bpc	= ½ $\angle$ bo2c = β.

The above method is sometimes known as the method of two intersecting circles.

(c) Third Method :

1. Join ab and bc.
2. At a and c, erect perpendiculars ad and ce.
3. At b, draw a line bd subtending angle (90° - α) with ba, to meet the perpendicular through a in d.
4. Similarly, draw a line be subtending an angle (90° - β) with bc, to meet the perpendicular through c in e.
5. Join d and e.
6. Drop a perpendicular on de from b. The foot of the perpendicular (i.e. p) is then the required position of the boat.

9. What are tides? Explain its types and formation.

All celestial bodies exert a gravitational force on each other. These forces of attraction between earth and other celestial bodies (mainly moon and sun) cause periodical variations in the level of a water surface, commonly known as tides. There are several theories about the tides but none adequately explains all the phenomenon of tides. However, the commonly used theory is after Newton, and is known as the equilibrium theory. According to this theory, a force of attraction exists between two celestial bodies, acting in the straight line joining the centre of masses of the two bodies, and the magnitude of this force is proportional to the product of the masses of the bodies and is inversely proportional to the square of the distance between them. We shall apply this theory to the tides produced on earth due to the force of attraction between earth and moon. However, the following assumptions are made in the equilibrium theory :

1. The earth is covered all round by an ocean of uniform depth.
2. The ocean is capable of assuming instantaneously the equilibrium , required by the tide producing forces. This is possible if we neglect (i) inertia of water, (ii) viscosity of water, and (iii) force of attraction between parts of itself.

1. The Lunar Tides

(a) shows the earth and the moon, with their centres of masses $O1$ and $O2$ respectively. Since moon is very near to the earth, it is the major tide producing force. To start with, we will ignore the daily rotation of the earth on its axis. Both earth and moon attract each other, and the force of attraction would act along $O1O2$. Let O be the common centre of gravity of earth and moon. The earth and moon revolve monthly about O, and due to this revolution their separate positions are maintained. The distribution of force is not uniform, but it is more for the points facing the moon and less for remote points. Due to the revolution of earth about the common centre of gravity O, centrifugal force of uniform intensity is exerted on all the particles of the earth. The direction of this centrifugal force is parallel to $O1O2$ and acts outward. Thus, the total force of attraction due to moon is counter-balanced by the total centrifugal force, and the earth maintains its position relative to the moon. However, since the fore of attraction is not uniform, the resultant force will very all along. The resultant forces are the tide producing forces. Assuming that water has no inertia and viscosity, the ocean enveloping the earth's surface will adjust itself to the unbalanced resultant forces, giving rise to the equilibrium. Thus, there are two lunar tides at A and B, and two low water positions at C and D. The tide at A is called the superior lunar tide or tide of moon's upper transit, While tide at B is called inferior or antilunar tide.

Now let us consider the earth's rotation on its axis. Assuming the moon to remain stationary, the major axis of lunar tidal equilibrium figure would maintain a constant position. Due to rotation of earth about its axis from west to east, once in 24 hours, point A would occupy successive position C, B and D at intervals of 6 h. Thus, point A would experience regular variation in the level of water. It will experience high water (tide) at intervals of 12 h and low water midway between. This interval of 6 h variation is true only if moon is assumed stationary. However, in a lunation of 29.53 days the moon makes one revolution relative to sun from the new moon to new moon. This revolution is in the same direction as the diurnal rotation of earth, and hence there are 29.53 transits of moon across a meridian in 29.53 mean solar days. This is on the assumption that the moon does this revolution in a plane passing through the equator. Thus, the interval between successive transits of moon or any meridian will be 24 h, 50.5 m. Thus, the average interval between successive high waters would be about 12 h 25 m. The interval of 24 h 50.5 m between two successive transits of moon over a meridian is called the tidal day.

2. The Solar Tides

The phenomenon of production of tides due to force of attraction between earth and sun is similar to the lunar tides. Thus, there will be superior solar tide and an inferior or anti-solar tide. However, sun is at a large distance from the earth and hence the tide producing force due to sun is much less.

Solar tide = 0.458 Lunar tide.

4. Combined effect : Spring and neap tides

Solar tide = 0.458 Lunar tide.

Above equation shows that the solar tide force is less than half the lunar tide force. However, their combined effect is important, specially at the new moon when both the sun and moon have the same celestial longitude, they cross a meridian at the same instant.

Assuming that both the sun and moon lie in the same horizontal plane passing through the equator, the effects of both the tides are added, giving rise to maximum or spring tide of new moon. The term 'spring' does not refer to the season, but to the springing or waxing of the moon. After the new moon, the moon falls behind the sun and crosses each meridian 50 minutes later each day. In after 7 ½ days, the difference between longitude of the moon and that of sun becomes 90°, and the moon is in quadrature . The crest of moon tide coincides with the trough of the solar tide, giving rise to the neap tide of the first quarter. During the neap tide, the high water level is below the average while the low water level is above the average. After about 15 days of the start of lunation, when full moon occurs, the difference between moon's longitude and of sun's longitude is 180°, and the moon is in opposition. However, the crests of both the tides coincide, giving rise to spring tide of full moon. In about 22 days after the start of lunation, the difference in longitudes of the moon and the sun becomes 270° and neap tide of third quarter is formed. Finally, when the moon reaches to its new moon position, after about 29 ½ days of the previous new moon, both of them have the same celestial longitude and the spring tide of new moon is again formed making the beginning of another cycle of spring and neap tides.

4. Other Effects

The length of the tidal day, assumed to be 24 hours and 50.5 minutes is not constant because of (i) varying relative positions of the sun and moon, (ii) relative attraction of the sun and moon, (iii) ellipticity of the orbit of the moon (assumed circular earlier) and earth, (v) declination (or deviation from the plane of equator) of the sun and the moon, (v) effects of the land masses and (vi) deviation of the shape of the earth from the spheroid. Due to these, the high water at a place may not occur exactly at the moon's upper or lower transit. The effect of varying relative positions of the sun and moon gives rise to what are known as priming of tide and lagging of tide.

At the new moon position, the crest of the composite tide is under the moon and normal tide is formed. For the positions of the moon between new moon and first quarter, the high water at any place occurs before the moon's transit, the interval between successive high water is less than the average of 12 hours 25 minutes and the tide is said to prime. For positions of moon between the first quarter and the full moon , the high water at any place occurs after the moon transits, the interval between successive high water is more than the average , and tide is said to lag. Similarly, between full moon and 3rd quarter position, the tide primes while between the 3rd quarter and full moon position, the tide lags. At first quarter, full moon and third quarter position of moon, normal tide occurs.

Due to the several assumptions made in the equilibrium theory, and due to several other factors affecting the magnitude and period of tides, close agreement between the results of the theory, and the actual field observations is not available. Due to obstruction of land masses, tide may be heaped up at some places. Due to inertia and viscosity of sea water, equilibrium figure is not achieved instantaneously. Hence prediction of the tides at a place must be based largely on observations.

mean sea level ? Explain why it is used as datum.

For all important surveys, the datum selected is the mean sea level at a certain place. The mean sea level may be defined as the mean level of the sea, obtained by taking the mean of all the height of the tide, as measured at hourly intervals over some stated period covering a whole number of complete tides, The mean sea level, defined above shows appreciable variations from day to day, from month to month and from year to year. Hence the period for which observations should be taken depends upon the purpose for which levels are required. The daily changes in the level of sea may be more. The monthly changes are more or less periodic. The mean sea level in particular month may be low while it may be high in some other moths. Mean sea level may also show appreciable variations in its annual values. Due to variations in the annual values and due to greater accuracy needed in modern geodetic levelling, it is essential to base the mean sea level on observations extending over a period of about 19 years. During this period, the moon's nodes complete one entire revolution. The height of mean sea level so determined is referred to the datum of tide gauge at which the observations are taken. The point or place at which these observations are taken is known as a tidal station. If the observations are taken on two stations, situated say at a distance of 200 to

500 kms on an open coast, one of the station is called primary tidal station while the other is called secondary tidal station. Both the stations may then be connected by a line of levels.

Cartographic concepts and techniques.

Cartography is about maps. This includes the art, science and technology of map making, the use of maps as research tools and as sources of information, and the study of maps as historical documents and works of art.

Definition of Cartography

Cartography has always been closely associated with Geography and Surveying. Its recognition as a distinct discipline is relatively recent. Scientific journals dealing with Cartography began to appear in the middle of the twentieth century. Numerous definitions of Cartography have appeared in the literature. Earlier definitions tend to emphasize map making while more recent definitions also include map use within the scope of Cartography.

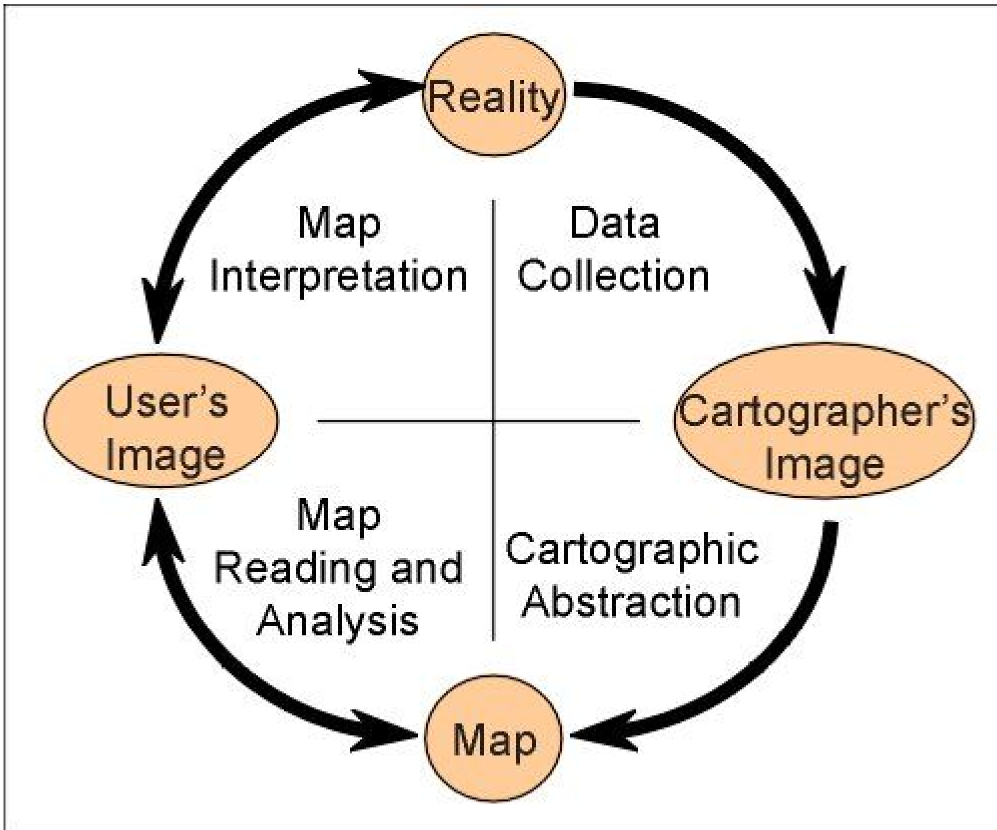
More often than not, the map user is different from the map maker and the map maker rarely collects the original data. Recognition of this has led to a redefinition of Cartography which is based in part on work by communications theorists. In this context, Cartography is viewed as being concerned with a particular form of communications process which relies on graphic images, i.e. maps, to convey information about data and the spatial relationships between them, e.g. a geographic environment. The cartographer is the map maker. The map is the communications medium. The data may be about towns, temperatures, bedrock, people, crops, water depths, algae growth patterns, the stars, or even about cellular structure, neural networks, or DNA. The map represents the spatial relationships among the individual pieces of data. The map user "reads" the map and interprets its information content in the context of his or her own objectives and knowledge of the environment or spatial pattern which the map describes.

Scope of Cartography

Cartography is the art, science and technology of map making and map use, and the study of maps in all its aspects.

Cartographic Communication

Several models of cartographic communication have been proposed. While differing in detail, the models share a common recognition of the separation between map making and map use but emphasize the close relationship between these processes by treating them as components of a communication system. Effective use of maps requires understanding of the nature of maps and the mapping process while good map design requires understanding of how the maps will be used. The communications model also serves to emphasize that map use is not simply the reverse of map making but requires a distinct set of skills.Cartographic Communication Process



Cartographic communication is a special form of graphic communication which differs from verbal communication. Verbal communication is sequential. Ideas are presented in a sequential fashion, allowing the writer or speaker to control the order in which information is conveyed. The emphasis is on parts linked by logic. In contrast, maps are synoptic, presenting information in a holistic fashion. The map user receives all of the information at once. Thus the map maker is unable to control the order in which information is received except by using map design to emphasize the most important information.

Cartographic Processes

The communications model of Cartography emphasizes that maps are used by a wide range of users for many different purposes. The role of the cartographer in the communications process is primarily associated with map making. However, the cartographer must have a good understanding of the subject matter of the map as well as a good understanding of how the map is likely to be used in order to design an effective map.

Map Making

While there are many steps involved in the map making process, they can be grouped into three main stages: data collection, organization, and manipulation; map design and artwork preparation; and map reproduction.

1) Collection, Organization and Manipulation of Data
Data must be collected from existing maps, aerial photographs or digital imagery, documents, e.g. legal descriptions of property boundaries, historical documents, etc., field work or questionnaire surveys. The data must be organized so that we can understand whatever phenomena are being represented and the data must be manipulated into a form which is suitable for mapping. This may involve aggregating data to some specified set of spatial units, calculating percentages, densities or other summary measures from the raw data.

Example. When Champlain crossed the Lake Ontario basin in 1615 he was accompanied by French Cartographers (people who collected data to make maps - we would now call them surveyors). They recorded: offshore soundings, the condition of inlets and potential harbours, rivers and their navigability, and shoreline conditions. Other types of data such as roads or trails and native settlements were not shown since they wanted to represent Canada as a pristine environment waiting to be settled.

Data have to be organized and simplified to ensure effective communication. If the map contains too much detail it will be difficult to read and understand so it is not a good map. But it must contain enough detail to get the required idea across. What you leave out is often as important as what you put in.

Example Even using topographic data collected by surveyors, the cartographer still has to decide which contours will be included on the map. For a map of Denmark or the Netherlands where most land is below 50m, a contour interval of 5m or perhaps even 1m might be appropriate whereas in the Himalayas, with elevations of up to 8000m, a much larger contour interval would be required to produce a legible map.

2) Design and Preparation of Maps, Charts, Plans and Graphs
Many decisions go into the design of an effective map. These include the selection of the geographic features and thematic attributes to be represented on the map. These choices depend upon the purpose of the map, the intended audience, and the cartographer's understanding of the phenomena being represented. For maps of large areas such as provinces, countries, continents or the world, it is important to choose an appropriate map projection which minimizes distortion of the geometric properties of the region being mapped. Determination of the level of detail required, given the purpose of the map, is a critical decision which is closely related to the choice of map scale. A small scale map can show a large area but little detail while a large scale map shows a smaller area but with more detail.

Example. Road maps can be produced at a variety of scales. A road map of Ontario might use a scale of 1: 750,000 and show highways and major regional roads. A road map of Kitchener-

Waterloo might use a scale of 1:25,000 and show all roads as double line rather than single line features.

Different scales imply different degrees of generalization of the features shown on the map. For example, if you compare the contour lines on a map at a scale of 1:250,000 with those on a map of the same area at a scale of 1:50,000, you will find that contours on the two maps are quite different. The 1:250,000 scale map will use a larger contour interval, have fewer contour lines, and will simplify the shapes of complex features. Minor features of the landscape may be omitted altogether. Thus in using maps, it is important to consider what level of detail is appropriate and therefore what scale of map is needed.

Designing the map also includes consideration of how the information will be symbolized. Do you show the data in colour or not? Can you afford to reproduce the map in colour? Are the data qualitative or quantitative? Do the data represent a continuous, stepped or discrete surface? Will you represent the data using point, line or area symbols? How will you arrange the map itself as well as items such as title blocks, legends, and scale symbols on the page?

Example . For examples of well designed and well executed maps, see the Historical Atlas of Canada produced by Matthews who is a cartographer at the University of Toronto. Matthews is a true creative genius who worked with a group of academics to produce a series of essays about different periods of Canadian history from 12,000 B.C. to the present in three volumes. They didn't just write the essays but they mapped all the related data. Very, very hard! Think about it. You are asked to write an essay on the fur trade, immigration, housing styles, or French expansion. Then you are asked to produce a map instead of an essay but it must explain the same thing. The result makes fascinating reading for anyone interested in Cartography or Canadian history.

3) Map Reproduction

Map reproduction methods act as a constraint on the map design process. How many copies of the map will be required? This is the major determinant of the reproduction methods used. Black and white and colour laser printing and Xeroxing technology are cost effective if only a few copies are required. If large numbers of maps are required then offset printing may be the only practical alternative. In some instances, distribution of maps in digital format on tape, disk or CD-ROM is replacing or at least reducing the need for printed maps.

Map Use

Map use is a learned process which requires use of a variety of skills. In order to use maps effectively, you must understand the rules and conventions governing cartographic representation of information. Otherwise, it will be difficult to extract meaning from maps.

Cartographic Steps

Step 1	Consider what the real world distribution of the phenomenon might look like
Step 2	Determine the purpose of the map and its intended audience
Step 3	Collect data appropriate for the map's purpose
Step 4	Design and construct the map
Step 5	Determine whether users find the map useful and informative

The recent trends in cartography.

Web cartography can be considered a trend in cartography. However there are other recent trends that affect cartography and the way web cartography is developing. These have to do with the impact of visualisation and the need for interactivity and dynamics as well as the widespread use of geographical information systems resulting in many more maps being produced by many more people. In the context of geospatial data handling, the cartographic visualisation process is considered to be the translation or conversion of geospatial data from a database into map-like products. This process is guided by the saying "How do I say what to whom, and is it effective?"

The above developments have given the word visualisation an enhanced meaning. According to the dictionary, it means 'make visible' and it can be argued that this has always been the business of cartographers. However, progress in other disciplines has linked the word to more specific ways in which modern computer technology can facilitate the process of 'making visible' in real time. This results in visualisation for presentation and exploration. Presentation fits into the traditional realm of cartography, where the cartographer works on known geospatial data and creates communicative maps. These maps are often created for multiple uses. Exploration, however, often involves a discipline expert creating maps while dealing with unknown data. These maps are generally for a single purpose, expedient in the expert's attempt to solve a problem. While dealing with the data, the expert should be able to rely on cartographic expertise, provided by the software or some other means.

In the past cartography played an important role in the exploration of the world. Maps were used to chart unknown territories. A new phase in mapping the unknown has recently started. This does not refer to the cartographic or geographic exploration discussed in the previous paragraph. It deals with the mapping of cyberspace.