Technical College/ Kirkuk

Surveying Engineering Department

Surveying Instrument Maintenance

Subject Lecture : Dr. Qahtan Ahmed

قسم هندسة تقنيات المساحة

منهاج المرحلة الدراسية الرابعـــة **المادة / صيانــة الأجهـــزة المساحيـــة**

أسم المقـــرر	الساعات النظريـــة	الساعات العملية	عدد الوحدات
صيانــة الأجهـــزة المساحيـــة	2	4	4

منـهاج المادة النظرية و العمليـــة **عدد الأسابيـــع / 30 أسبوعا"**

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عنوان المحاضرة :

Accuracy and Error diffinition and their using in surveying operation.Kinds of error

Accuracy and Error الدقة والخطأ :-

In dealing with measurements, it is important to distinguish between *accuracy* and *precision*.

As defined by the American Society of Civil Engineers, *Accuracy* is ((absolute nearness to the truth)), where as *precision* is ((relative or apparent nearness to the truth)).

As defined by the U.S Coast and Geodetic Survey, accuracy is ((degree of conformity with a standard)), where as precision is ((degree of refinement in the performance of an operation or in the statement of a result)).

From these mutually ,consistent definitions is follows that a measurement may be accurate without being precision and vice versa ,for example ; a distance may be measured very carefully with a tape , to thousandths of a foot ,and still be in error of several hundred of a foot because of errorness ength of tape ,the measurement is precise but not accurate.

Errors الأخطاء :-

True error is defined as the difference between the measured value and true value.
As the true value for any quantity is seldom known, the true error is seldom determined.

Discrepancy:-

Is the observed difference between two similar measurements, each of which contains some error which may or may not be appreciable.
Discrepancy does not indicate the magnitude of error but only indicates the observed difference.

Kinds of error أنواع الأخطاء :-

1) ***A mistake or blunder: -*** are the personal errors of the observers caused mainly due to carelessness and inexperience, they are detected and eliminated by checking all work.
2) ***Systematic errors الأخطاء النظامية :-*** a rise from known sources and generally can be eliminated if the cause is detected ,imperfect length of the tape or chain ,lack of adjustment of the instruments ,etc ; are sources of systematic errors.
3) ***Accidental errors الأخطاء العارضة :-*** imperfection of the instruments and limitation of human sight in observing and reading the true values are examples of accidental errors.

Errors may be of the compensating or cumulative type. In a series of measurement or observations when it is found that errors change their sign it is called compensating type of errors.
Accidental errors are normally found to be of the compensating type.
Cumulative errors are those where in a series of measurement, the errors do not change their sign. In a linear measurement when a tape which is shorter or longer than the standard is used, the errors are always of the same sign and produce the cumulative errors.

<u>الأسبـــوع الثـانــــي</u>

عنوان المحاضرة:-

The classification of errors during **surveying operation**

<u>The classification of errors تصنيف الأخطاء :-</u>

1) ***<u>Blunders الأخطاء :-</u>*** are gross errors whose value is greater than can be expected under particular conditions of measurements, given a definite observer, instrument, measurement procedure and natural conditions.

These are generally due to the carelessness of the observer they may include reading the wrong end of a tape, the wrong count of the degrees or minutes in angular measurement; etc. it is possible to track down gross errors by conducting control measurements, the results that contain errors are cancelled and measurements are repeated to permit correct results to be obtained.

2) ***<u>Constant errors الأخطاء الثابتة :-</u>*** ones that result from definite sources and under given condition show a definite sign and are of a definite absolute value and occur systematically ,the effect of constant errors may be expressed though a function.

Constant errors may include an error in staff readings induced by non-parallelism of the axis of a cylindrical bubble tube and collimation line of a geodetic levels telescope, an error in the length due to the difference in temperatures of the measuring tools.

Constant errors represent the greatest hazard to measurements, care should be taken to reduce their effect on the measured results.

There are <u>two</u> ways to do that:-

i. To give corrections to the measured results for length of a tape, for the temperature of the measuring tool.

ii. To use the requisite measurement procedure such as to rule out constant errors from the measured results, e.g.; by measuring horizontal angles twice with the telescope direct and reversed then the average angle value proves free from a collimation error, set up the level in between the staffs, if it is exactly at the middle of the distance, the calculated height difference will be free from the error caused by the non-parallelism of the collimation line.

3) ***Random errors الأخطاء العشوائية :-*** are such errors the effect of which in each particular case cannot be predicted ,under given conditions , they are as likely as not to occur ,be great or small , positive or negative ,random errors follows a definite regularity provided that a great number of measurements have been made ,they cannot be predicted in advance or taken in to account because of their very nature ,they cannot be avoided during measurements ,example of random errors ,an error of interpolating during taking readings from E –pattern leveling staff ,an error in taking sights by a theodolites telescope.

As to random errors, they will occur, and there is no way of excluding them from measurements. However it is plausible to try to possibly reduce the effect of random errors on measured results, the theory of errors is used to establish of unavoidable random errors, reduce their effect as much as possible and determine the limiting errors tolerated in each process of measurement.

الأسبــوع الثالث

عنوان المحاضرة:-

Method of Least Square principle, a problem of conditional extremum and the method of solution consists in constructing a new auxiliary function

Method of Least Square طريقة المربعات الصغرى :-

The principle of least squares states that the most probable value of an observed quantity for a set of observation is that for which the sum of the squares of the residual errors is minimum.

Suppose we are investigating a function

$$Z = Z(e_1, e_2, \ldots, e_n) = a_1e_1 + a_2e_2 + a_3e_3 + \ldots, a_ne_n \qquad (5.20)$$

For an extremum, i.e., the minimum value in the method of least squares when e_1, e2 …. , en are errors of observation, subject to the following constrains.

$$f_i\,(e_1 + e_2 + \ldots + e_n) = 0\ , (I = 1, 2 \ldots, m ; m < n) \qquad (5.21)$$

The constraints arise out of certain condition equations involving the errors (e_1, e_2, … , e_n). This is a problem of conditional extremum and the method of solution consists in constructing a new auxiliary function

$$Z^* = Z + \sum_{i=1} \lambda_i f_i \qquad (5.22)$$

Where λ_i are certain constant factors normally known as Lagrange's multipliers.

The auxiliary function Z^* (e_1, e_2, … ,e_n) of eq. (5.22) is now investigated for an unconditional extremum in the usual manner by setting up the following system of equation:-

$$\frac{\partial Z^*}{\partial e f} = 0\ , \upsilon = 1, 2, n) \qquad (5.23)$$

Supplemented by the constraint equations

$f_i = 0$, $(i = 1 , 2 \ldots m; m < n)$ (5.24)

Thus from eqs ,(5.23) we have $n + m$ equations for finding out $n + m$ unknown : e_1 , e_2 , e_3 , … , e_n and λ_1 , λ_2 , … ,λ_m.

Lets us illustrate the aforementioned method with a simple but general example in theory of errors. Suppose there is a level circuit having a closing error *E*. The weights attached to the observations of the differences of levels of the several benchmarks are w_1 , w_2 , … ,w_n while e_1 , e_2 , … , e_n are the necessary corrections to the observed level differences.

Now, the least square condition requires that

$$Z = w_1e_1^2 + w_2e_2^2 + \ldots. + w_ne_n^2 \qquad (5.25)$$

Is a minimum while one equation of condition to serve at the constraint is

$$f = (e_1 + e_2 + e_3 + \ldots + e_n \pm E) = 0 \qquad (5.26)$$

Hence from (5.22) the auxiliary function Z* is

$$Z^* = (w_1e_1^2 + w_2e_2^2 + \ldots. + w_ne_n^2) + \lambda(e_1 + e_2 + \ldots + e_n \pm E) = 0 \qquad (2.27)$$

Now, from eqs ,(2.23) and (5.24) , we have

$$\left.\begin{aligned}
\frac{\partial Z^*}{\partial e_1} &= 2w_1e_1 + \lambda = 0 \\
\frac{\partial Z^*}{\partial e_2} &= 2w_2e_2 + \lambda = 0 \\
\frac{\partial Z^*}{\partial e_3} &= 2w_3e_3 + \lambda = 0 \\
\frac{\partial Z^*}{\partial e_4} &= 2w_4e_4 + \lambda = 0 \\
\frac{\partial Z^*}{\partial \lambda} &= e_1 + e_2 + e_3 + e_4 \pm E = 0
\end{aligned}\right\} \qquad (2.28)$$

Hence,

$$-\frac{\lambda}{2}\left(\frac{1}{w_1}+\frac{1}{w_2}+\frac{1}{w_3}+\frac{1}{w_4}\right) \pm E = 0$$

or

$$\boxed{\lambda = \pm \frac{2E}{\frac{1}{w_1}+\frac{1}{w_2}+\frac{1}{w_3}+\frac{1}{w_4}}}$$

Thus, we get errors, e_1 , e_2 ,e_3 and e_4 by substituting λ in (5.28)

Weights:-

Sometimes some angles are measured more accurately than others. In a theodolites observation these can be achieved by changing the face of the instrument or changing the initial zero and taking the mean value.

Naturally such values carry more "weights" of observation of values x_1 , x_2 , x_3 , …. , x_n then the

$$\boxed{\text{weighted mean} = \frac{w_1x_1 + w_2x_2 + w_3x_3 + \ldots. + w_nx_n}{w_1 + w_2 + w_3 + w_4}} \qquad (5.4)$$

الأسبــوع الرابع

عنوان المحاضرة:-

Solution examples by using the method of least squares

Example (1):-

In a closed leveling survey starting from the bench mark (A) , the observed difference in level between the stations (A , B , C , D) are as given below :-

	Difference in level (m)	Weights of observation
Between A and B	$h_1 = +1.25$	$w_1 = 2$
Between B and C	$h_2 = -2.05$	$w_2 = 1$
Between C and D	$h_3 = -1.05$	$w_3 = 2$
Between D and A	$h_4 = +1.95$	$w_4 = 1$

By the method of least squares, calculate the probable difference in level between the stations.

Solution:-

Let the probable difference in level between

A and B = $h_1 + e_1$

B and C = $h_2 + e_2$

C and D = $h_3 + e_3$

D and D = $h_4 + e_4$

For a closed leveling survey, difference of level between A and A is zero.

Hence, the equation of condition surveying as constant in this case is

$e_1 + e_2 + e_3 + e_4 = -(1.25 - 2.05 - 1.05 + 1.95) = -0.1 = -E$

$e_1 + e_2 + e_3 + e_4 + 0.1 = 0$

The auxiliary function Z* will be:-

$Z^* = (w_1e_1^2 + w_2e_2^2 + w_3e_3^2 + w_4e_4^2) + \lambda\,(e_1 + e_2 + e_3 + e_4 + 0.1)$

For unconditional ectremum

$$\frac{\partial Z^*}{\partial e_1} = 2w_1e_1 + \lambda = 0 \longrightarrow 4e_1 + \lambda$$

$$\frac{\partial Z^*}{\partial e_2} = 2w_2e_2 + \lambda = 0 \longrightarrow 2e_2 + \lambda$$

$$\frac{\partial Z^*}{\partial e_3} = 2w_3e_3 + \lambda = 0 \longrightarrow 4e_3 + \lambda$$

$$\frac{\partial Z^*}{\partial e_4} = 2w_4e_4 + \lambda = 0 \longrightarrow 2e_4 + \lambda$$

$$\frac{\partial Z^*}{\partial \lambda} = e_1 + e_2 + e_3 + e_4 + E = 0$$

$$\lambda = + \frac{2E}{\frac{1}{w_1} + \frac{1}{w_2} + \frac{1}{w_3} + \frac{1}{w_4}} = \frac{2 \times 0.1}{\frac{1}{2} + 1 + \frac{1}{2} + 1} = 0.06666$$

Therefore, from –

$$e_1 = -\frac{0.0666}{4} = -0.0166$$

$$e_2 = -\frac{0.0666}{2} = -0.0333$$

$$e_3 = -\frac{0.0666}{4} = -0.0166$$

$$e_4 = -\frac{0.0666}{} = -0.0333$$

2

The probable difference in level between

A and B = 1.25 – 0.0166 = 1.2334

B and C = – 2.05 – 0.0333 = – 2.0833

C and D = – 1.05 – 0.0166 = – 1.0666

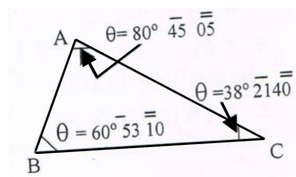
D and A = 1.95 – 0.0333 = 1.9167

Check difference in level between A and A = (0.0002) يهمل

<u>Example (2):-</u>

In a simple triangulation system the observed angles are:-

angle	weight of observation
$A = \theta_1 = +80^\circ\ 45'\ 05''$	$w_1 = 4$
$B = \theta_2 = +60^\circ\ 53'\ 10''$	$w_2 = 2$
$C = \theta_3 = +38^\circ\ 21'\ 40''$	$w_3 = 3$



What are the probable values of the angles?

<u>Solution:-</u>

Let e_1 , e_2 , e_3 be the error in the angles θ_1 , θ_2 , θ_3

Then the condition equation is,

$(\theta_1 - e_1) + (\theta_2 - e_2) + (\theta_3 - e_3) = 180^\circ$

$e_1 + e_2 + e_3 = \theta_1 + \theta_2 + \theta_3 - 180^\circ = -5''$

or $e_1 + e_2 + e_3 + 5'' = 0$

from least squares, the weighted sum of the squared error should be minimum

$w_1e_1^2 + w_2e_2^2 + w_3e_3^2 = 0$ (minimum)

Hence the auxiliary function is

$Z^* = (w_1e_1^2 + w_2e_2^2 + w_3e_3^2) + \lambda\,(e_1 + e_2 + e_3 + 5'')$

$$\frac{dz^*}{de_1} = 2e_1\,w_1 + \lambda = 0 = 8e_1 + \lambda$$

$$\frac{dz^*}{de_2} = 2e_2\,w_2 + \lambda = 0 = 8e_2 + \lambda$$

$$\frac{dz^*}{de_3} = 2e_3\,w_3 + \lambda = 0 = 6e_3 + \lambda$$

$$\frac{dz^*}{D\lambda} = e_1 + e_2 + e_3 + 5'' = 0$$

$$\lambda = \frac{10}{\frac{1}{4} + \frac{1}{2} + \frac{1}{3}} = \frac{120}{13} = 9.23''$$

$$e_1 = -\frac{9.23}{8} = -1.15''$$

$$e_2 = -\frac{9.23}{4} = -2.31''$$

$$e_3 = -\frac{9.23}{6} = -1.54''$$

The probable values of the angles are:-

A = 80˚ 45' 05" + 1.15" = 80˚ 45’ 6.15"

B = 60˚ 53' 10" + 2.31" = 60˚ 53' 12.31"

C = 38˚ 21' 40" + 1.54" = 38˚ 21' 41.54"

Check ∑ = 180˚ 00' 00"

<u>الأسبـــوع الخامس</u>

عنوان المحاضرة:-

Determination the instrument constant of a tacheometry by using the method of least squares

<u>Example:-</u>

In order to determine the instrument constant of a tacheometry from the expression $D = K_1S + K_2$

The following observations were made:-

Observation	D (m)	S (m)
1	15	0.155
2	30	0.305
3	45	0.455
4	60	0.600

What is the most probable value of the constants K_1 and K_2

<u>Solution:-</u>

If K_1 and K_2 are the most probable values of the constant, then the errors of observation are:-

For observation 1 : $15 - 0.155\ K_1 - K_2$

1 : $15 - 0.155\ K_1 - K_2$

2 : $30 - 0.305\ K_1 - K_2$

3 : $45 - 0.455\ K_1 - K_2$

4 : $60 - 0.600\ K_1 - K_2$

By the theory of least squares

$(15 - 1.515\,K_1 - K_2)^2 + (30 - 0.305\,K_1 - K_2)^2 + (45 - 0.455\,K_1 - K_2)^2 + (60 - 0.600\,K_1 - K_2)^2$ should be minimum

Differentiating of this equation with respect to K_1

$0.155\,(15 - 1.515\,K_1 - K_2) + 0.305\,(30 - 0.305\,K_1 - K_2) + 0.455\,(45 - 0.455\,K_1 - K_2) + 0.600\,(60 - 0.600\,K_1 - K_2) = 0$

or / $67.905 - 0.684\,K_1 - 1.515K_2 = 0$

Differentiating with respect to K_2

$(15 - 1.515\,K_1 - K_2) + (30 - 0.305\,K_1 - K_2) + (45 - 0.455\,K_1 - K_2) + (60 - 0.600\,K_1 - K_2) = 0$

Or / $150 - 1.515\,K_1 - 4K_2 = 0$

Then we get $K_1 = 100.6$

$K_2 = -0.6$

Problems:-

1) Observations were made from a station (O) by method of repetition to the stations A, B , C and D.

angle	observed value	weight
AOB	67° 14' 32.5"	3
BOC	75° 36' 21.3"	3
COD	59° 56' 02.2"	4
DOA	157° 13' 02.0"	5

Find the probable value of the angles? Ans.($\lambda = 3.582$)

2) Find the most probable value of the angles of the triangle ABC when the observed values are:-

angle	observed value	weight
ABC	49° 17' 23"	4
BAC	75° 32' 47"	5
ACB	55° 09' 52"	3

3) The following results were obtained for determining the instrument constant of a tacheometer by the formula $D = KS + K_1$. Compute the most probable value of the constants.

Distance (m)	15.0	22.5	30.0	60.0	100
(S) Staff intercept (m)	0.180	0.275	0.370	0.745	1.240

الأسبــوع السادس

عنوان المحاضرة:

Verniers **diffinition and their using in surveying operation ,giving some example in** Verniers reading ,Some mistakes commonly made in the use of the vernier transit

Verniers:-

Verniers are short auxiliary scales set parallel with and adjacent to primary scale, the vernier is so constructed that when it is placed so that both primary and vernier scales have a line in coincident position, the fractional part of the smallest of the primary scale can be obtained without interpolation.

In figure below, the vernier has n divisions is space covered by n – 1 of the smallest division on the scale. Thus:-

$$(n-1)\,d = nv \longrightarrow \boxed{v = \frac{(n-1)\,d}{n}}$$

d = the length of a main scale division.

v = the length of the vernier division.

Example: - in figure below; n = 10, d = 0.01 ft and v = 0.09 / 10 = 0.009 ft , what are the vernier reading?

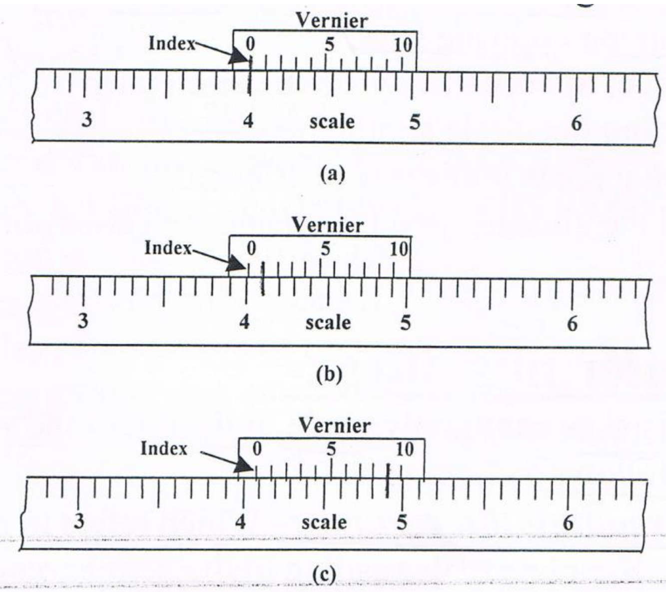


Fig 4.7 Vernier

The reading in vernier in (a) = 0.400

The reading in vernier (b) is equal d – v = 0.01 – 0.009 = 0.001 ft.

In notice that the vernier is moved so that its first graduation from zero coincides with the first graduation of the scale beyond

0.400 $\longrightarrow$ there 0.001 × 1 = 0.001 ft

Then the reading is equal 0.4 + 0.001 = 0.401 ft

The reading in (c)

The position where the vernier and primary scales are coincident, the vernier scale is at position of <u>8</u>. That means a reading of <u>8</u> indicates 0.008 ft then; the reading is equal 0.40 + 0.008 = 0.408 ft.

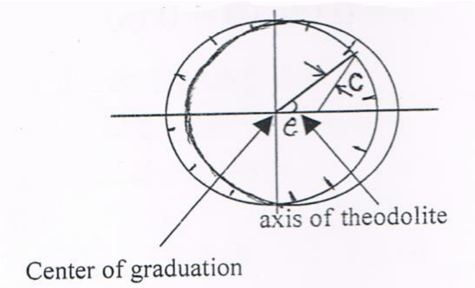
When a vernier is used, d – v is the smallest reading obtainable without interpolating, it is called the <u>least count</u> or <u>least reading</u> of the vernier.

$$\textit{Least count} = \frac{\textit{Value of the smallest division on the mean scale}}{\textit{Number of division on the vernier}}$$

To be certain the scale and vernier are being read correctly, on observer must determine the least count.

Eccentricity:-

When the circle is mounted on the theodolite, the center of the pattern should be exactly at the vertical axis of the alidade otherwise is called the eccentricity, which cause to reading too small angle in one side and too large on the opposite side.



The effect of eccentricity can be eliminated By reading the circle simultaneously at two or more indices uniformly distributed around the alidade , and then using the average.

Vernier mistake:-

The mistakes commonly made in the use of the vernier transit are as follows:-

1)Misreading the vernier :- which refers to forgetting to add the complete scale reading to the vernier reading , e.g. if the index reads 13˚ 30' and the vernier reads 06' the total reading is 13˚ 36' , but is sometimes mistakenly read as 13˚ 06'.

2) Reading the wrong vernier: - This refer to the mistake of reading the right –hand vernier when the left – hand one should be used.

3) Reading the wrong circle: - when the magnitude of an angle is near 180˚ it is a frequent mistake to read the wrong circle; i.e. the inner instead of the outer circle , or vice versa.

4) Reading the circle incorrectly: - the circle may be read incorrectly, especially for values near 10˚ divisions e.g. 59˚ is misread as 61˚, etc.

5) Using the wrong tangent screw: - A frequent mistakes with beginners is to use the wrong tangent motion i.e. the upper tangent screw with lower clamp or vice versa.

6) Recording: - this refers to the usual mistakes of recording, such as transposing numbers … etc.

The name optical transit will refer to instruments of moderate to high precision having glass circles which are read by viewing their graduations through either a microscope containing a fixed glass scale or a microscope with an optical micrometer.

The optical transit are characterized by an optical – reading system that permits the observer to obtain reading of both horizontal and vertical circles through an eyepiece situated beside the telescope.

Precision in theodolites survey:-

Precision in theodolites survey depends upon the purpose for which the survey is being conducted for ordinary surveys, such as for the location of roads, railway lines, etc. The angular error of the closer should not exceed $\underline{1' \times \sqrt{N}}$, where N is the number of angles, for surveys of important boundaries, such as city surveys, the multiplying constant is reduced to 30" instead of 1' and for very important surveys this should be farther reduced to 15".

الأسبـــوع السابع

عنوان المحاضرة:

Methods of measuring angles by theodolites, Measuring angles by repetition

Methods of measuring angles by theodolites:-

1) Measuring angles by repetition:-

A horizontal angle may be mechanically multiplied and the product can be read with the same precision as the single value.

The precision increase directly with the number of repetitions up to six or eight beyond this number by further repetition is not appreciable on account of lost motion in instrument and on account of accidental errors such as those due to setting the line of sight.

To repeat the angle, as AOB the transit is set up at O and the angle value of the angles is observed.

The vernier setting is left unaltered, the instrument is turned on its lower motion, and a second sight is taken to the first point, as A.

The upper clamp is loosened and the telescope is again sighted to B. the angle has been multiplied; the desired number of times of repetition can be done in this way the process.

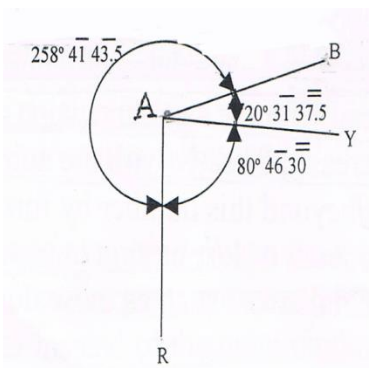
The vernier is read, and the value of the angle is determined by dividing the difference between initial and final reading by the number of times the angle was turned.

Usually the angle is multiplied four to ten times, half the observation being made with the telescope normal and half with it inverted, both Verniers are read and the mean values are used in the computations. Simple notes for measuring the angles about a point by repetition are shown in figure below.

For each angle, five repetitions are taken with telescope normal and five with telescope inverted.

Between station	Repetition	Vernier A	Vernier B	Vernier mean	Mean angle

B → Y	0	00° 00' 00"	180° 00' 00"	00° 00' 00"	
	1	20° 30' 00"			
	5	102° 38' 30"	282° 38' 30"		
	10	205° 16' 00"	25° 16' 30"	205° 16' 15"	20° 31' 37.5"
Y → R	0	205° 16' 00"	25° 16' 30"	205° 16' 15"	
	1	286° 03' 00"			
	5	249° 11' 00"	69° 11' 30"		
	10	293° 01' 00"	113° 01' 30"	293° 01' 15"	80° 46' 30"
R → B	0	293° 01' 00"	113° 01' 30"	293° 01' 15"	
	1	191° 43' 00"			
	5	149° 32' 30"	326° 33' 00"	359° 58' 30"	
	10	359° 58' 30"	179° 58' 30"	359° 58' 30"	258° 41' 43.5"



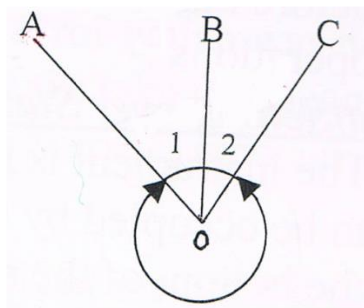
<u>الأسبـــوع الثامن</u>

عنوان المحاضرة:

Methods of measuring angles by theodolites, Method of reiteration

<u>2) Method of reiteration:-</u>

When there are more than one angle at a given station , the method of repetition becomes too cumbersome and is not useful , for example if at a station O the various angles are to be measured < AOC , < BOC , the method of reiteration is adopted.



The vernier is clamped to <u>zero</u> or initial reading in noted. Loosen the lower clamp , bisect station A , loosen the upper clamp , bisect station <u>B</u> and take vernier readings , the difference given angle <u>AOB</u>.

The upper clamp is loosened again, station <u>C</u> is bisected, and the reading noted again, the difference in values from previous reading gives angle <u>BOC</u>. From the last station, the instrument is swung to close the horizon; the closing error should be within permissible limits.

Three different settings of the vernier are taken on each face , i.e. , 0 , 90 , 180 , on face left , the instrument is swung in the right direction and vice versa , the mean of twelve observation of each angle gives the correct horizontal angle.

Horizontal Angles Measurement By Method Reiteration

Face left   swing right   face right   swing left

Instrument station	station bisected	Vernier A ° ' "	Vernier B ' "	Horizontal angle ° ' "	Vernier A ° ' "	Vernier B ' "	Horizontal angle ° ' "	Mean Horizontal angle ° ' "	Total average Angle ° ' "
0	A	00 00 00	00 00		00 00 00	00 00			
				20 10 20			20 10 10	20 10 15	<AOB= 20 10 10.5
1st station	B	20 10 20			20 10 00	10 20			
				30 10 20			30 10 20	30 10 20	<BOC= 30 10 18.5
Close	C	50 20 40			50 20 20	20 40			
Horizon	A	00 00 20			00 00 20	00 00			
Second	A	90 00 00	00 20		90 00 00				
Setting				20 10 00		00 00	20 10 20	20 10 10	
	B	110 10 00	10 20		110 20 10	10 00			
				30 10 20			30 10 20	30 10 20	
Close	C	140 20 20	20 40		140 20 20				
Horizon	A	90 00 00			90 00 20	20 20			
Third	A	180 00 00	00 00		180 00 00	00 20			
Setting				20 10 00			20 10 20	20 10 10	
	B	200 10 00	00 00		200 10 20	10 40			
				30 10 20			30 10 20	30 10 15	
Close	C	230 20 20	10 20		230 20 40	20 40			
horizon	A	180 00 20	20 20		180 00 20				

الأسبـــوع التاسـع

عنوان المحاضرة:

Adjustment of Theodolites ,Temporary Adjustment

Adjustment of Theodolites:-

1)Temporary Adjustment:-

Those adjustments that are required to be made at each setting before making observation, this process involves the following operations:-

a)Setting over station :-

the instrument is fixed on the tripod which placed on the station to be occupied by the instrument , a plumb bob is suspended at the bottom of the axis and the centering is done by moving the legs radially till is approximately centered over the station mark.

b)Leveling the instrument :-

Bring all the foot screws in the middle of their run before using them for leveling. Then bring the bubble parallel to any two foot screws and bring it in the center by moving both the screws either in or out. Turn the theodolite through (90˚) till the plate level is parallel to the third foot screw. Bring the bubble to the center with the help of the third foot screw. The position of the instrument should be the same as it was in the first setting , again take the bubble to (90˚) and bring it in the center , in both these two positions the bubble should be remain in the center.

c)Removing parallax :-

Place a piece of white paper in front of the objective to cut off some of the bright light. Turn the eye piece around its own axis till the cross wires are sharply visible. Then direct the telescope to a far – off point. Move the focusing screw till a clear image is formed, after these operations the instrument is ready for taking observations.

الأسبـــوع العاشر

عنوان المحاضرة:

Adjustment of Theodolites, Permanent adjustment, First adjustment: - to make the axis of the plate levels perpendicular to the vertical axis

2)Permanent adjustment

From the continued use of instrument, many defects developed. The surveyor must know the nature and the relative importance of the defects produced due to these defects for successful operation of the theodolites. Provisions have been made for the adjustments of the various parts so as to eliminate the serious type of errors , also some defects come from the nonadjustable parts and defects can reduced by the instrument should be stable , it should not have any slackness and checking that all nuts and fixture of the clamp are tight.

In the testing and eliminating errors in the adjustable parts of the theodolites , the principle of reversal is used thus changing the face of the theodolite , during reversal , the error which was not apparent earlier will become apparent , so if we take the mean of the observations before and after doing the reversal , most errors are eliminated.

First adjustment: - to make the axis of the plate levels perpendicular to the vertical axis.

If the adjustment is correct, the plate bubble will remain in the center even after rotation to any degree about the vertical axis. So once the bubble is brought to the center it will remain in the center during its operation in that setting.
To check it, set the instrument on firm ground after fixing the lower clamp, level the bubble carefully swing the upper plate through (180°) , if the bubble remains in the center, the adjustment is correct, if not, note the deviation of the bubble and bring it back half way by means of two capstan – headed screws fixed at the end of tube, the remaining half should be adjusted with the help of the leveling screws, repeat the test till the instrument is perfectly adjusted.

<u>الأسبــوع الحادي العاشر</u>

عنوان المحاضرة:

Second adjustment: -_to make the line of means that the intersection of the cross hair should coincide the horizontal and vertical hairs are checked, Horizontzl hair adjustment, Vertical hair adjustment

<u>Second adjustment: - </u>to make the line of means that the intersection of the cross hair should coincide the horizontal and vertical hairs are checked.

<u>Adjustment of Horizontal Hair:-</u>

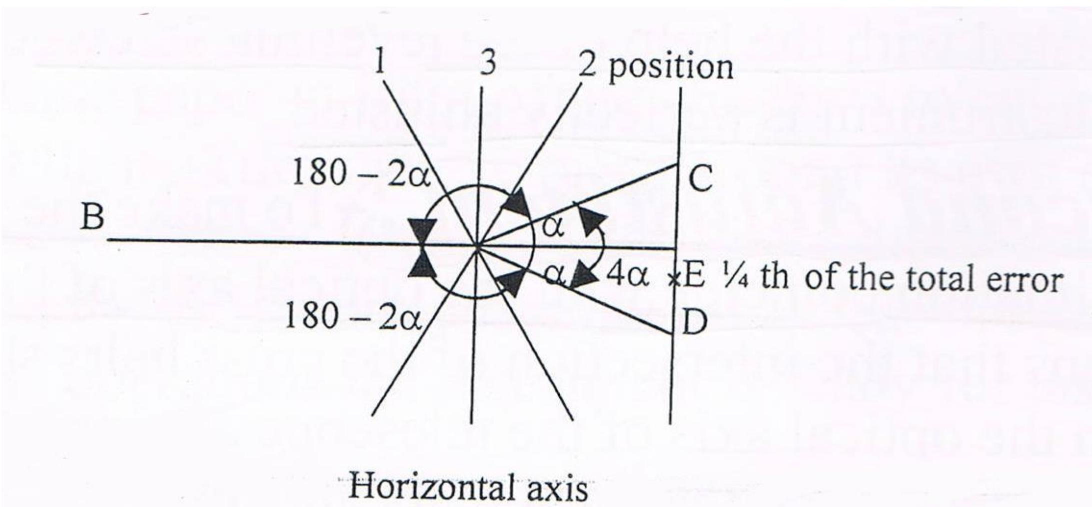
The necessary of this adjustment is more predominant in external – focusing telescopes in which the objective moves during focusing. So the line of <u>sight</u> will have varying direction at different settings.

Set up and level the instrument carefully. Clamp the vertical circle to zero and take a staff reading on firm ground. Unclamp and transit the telescope and swing it through (180˚). Set the vertical circle again to the <u>zero</u> position and take reading, if the previous reading is obtained, the adjustment is correct, otherwise find the mean position and adjust the horizontal hair by means of the top and bottom <u>diaphragm nuts</u> of the telescope. Repeat the test till there is no error after changing the faces.

<u>Adjustment of Vertical Hair:-</u>

It means placing the line of collimation perpendicular to the horizontal axis, then when the telescope is moved up or down, it will generate a single vertical plane on both the faces if the adjustment is correct, otherwise a cone will be generated with the vertex at the instrument center.

To check the adjustment, set the instrument on a fairly level ground and place it on such a position that clear sights are available on either side, sights an arrow fixed at point (B) on one side and clamp the horizontal movements, transit the telescope and mark a point (C) in line and almost at the same distance. Change the face and again sight the point (B) , clamp the horizontal movement and transit the telescope once again. If the line if sight now again intersects (C) , the adjustment is corrected, otherwise fix a point one – fourth of the total deviation from the last point (D) towards the first point and adjust the vertical hair by means of diaphragm nuts.



الأسبـــوع الثاني عشر

عنوان المحاضرة:

Third adjustment:- To make the horizontal axis perpendicular to the vertical axis .
Forth adjustment:-To make the bubble – tube axis of the telescope parallel to the line of collimation

Third adjustment:-

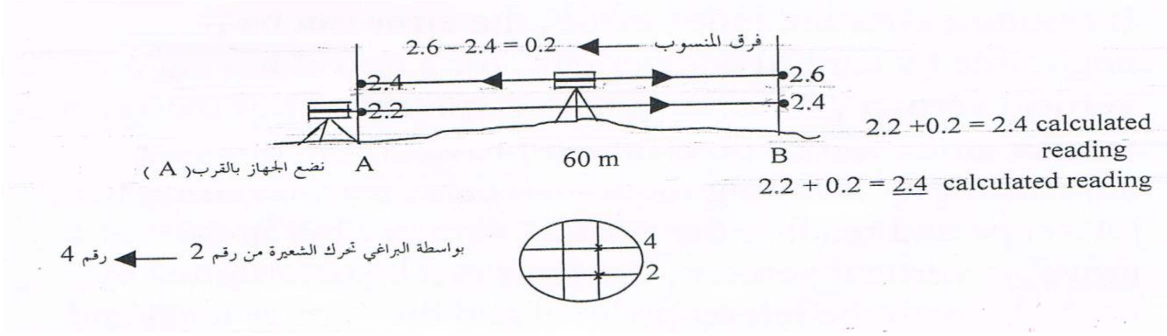
To make the horizontal axis perpendicular to the vertical axis. The line of sight should move in a vertical plane when the telescope is moved up or down, this adjustment is very essential in works involving the motion of the telescope in altitude. Set the instrument in position such that a well defined, high, sharp point is available for sighting. Bisect that point with both clamps fixed. Depress the telescope and mark a point on the ground. Change the face and again set the telescope on the same high point. Depress the telescope once again with horizontal movement clamped; if the same point is obtained on the ground, the adjustment is correct.

Otherwise , locate a point at the center of the two and set cross – hairs to bisect this point by supporting nuts of trunnion axis by raising or lowering the adjustable end of the axis , this test is also called the spire test because normally a high spire on the top of a building is selected.

Forth adjustment:-

To make the bubble – tube axis of the telescope parallel to the line of collimation , to test this adjustment , the two – peg is adopted as done in the case of dumpy level , select two stations (A) and (B) on level ground about (60 m) apart , set the instrument exactly at the mid – point. Find the true difference in level between the two points by clamping the vertical circle to zero and bring the telescope bubble to the center by means of the clip screw before taking staff readings, shift the instrument near one of the points and level the tube accurately, take a staff reading on the nearer point, compute the reading on the far point by adding or subtracting the true rise or fall.

Take the staff reading on the far point, if the computed reading is obtained, the adjustment is correct. Otherwise bring the horizontal hair to the computed reading by means of clip screw, since the vernier was already clamped to zero, there will be no index error.



الأسبـــوع الثالث عشر

عنوان المحاضرة:

The Error Of Theodolite Instrument

The Error Of Theodolite Instrument:-

Index error is error in an observed angle due to:-

1) Lack of parallelism between the line of sight and the axis of the telescope level.

2) Displacement (lack of adjustment) of the vertical vernier.

3) For a transit having a fixed vertical vernier , inclination of the vertical axis.

If the instrument were in perfect adjustment and were leveled perfectly for each observation there would be no index error. The effect of index errors due to lack of adjustment of the instrument can be eliminated by double – sighting for each observation.

1)Lack of parallelism between line of sight and axis of telescope level

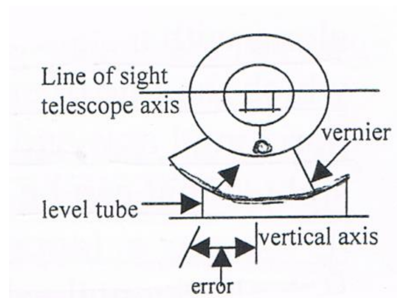
The error in vertical angle results from not parallelism can be rendered negligible for ordinary work by careful adjustment of the instrument as explained in (adjustment No. 4) , the index error due to the two causes (not parallelism and displacement of vertical vernier) can be determined by comparing a single reading on any given point with the mean of two readings obtained by double – sighting to the same point.

Thus, if the observed vertical angle to a point is + 2˚ 58' 30'' with telescope normal and + 2˚ 55' 30'' with telescope reversed, the index error for reading with telescope normal is

$(2^\circ\ 58'\ 30'' - 2^\circ\ 55'\ 30'') / 2 = +1'\ 50''$

2)Displacement of vertical vernier:-

It results a constant index error, the error can be rendered negligible by careful adjustment, for a transit having a fixed vertical vernier, the error due to displacement of the line of sight is in adjustment by leveling the transit carefully, leveling the telescope and reading the vertical vernier but in case of a movable vertical vernier, the error can be determined by leveling both the telescope level and the vernier level and reading the vertical vernier.



3)Inclination of vertical axis:-

The inclination introduces error resulted by error in leveling of the instrument , this error varies with the direction in which the telescope is pointed , this error can be rendered negligible by careful leveling of the transit before each observation , making sure that the plate – level bubbles remain in position for any direction of pointing , it is not eliminated by double sighting , if the line of sight and vertical vernier are in adjustment , the index error due to inclination of the vertical axis alone can be determined for each direction of pointing by leveling the telescope and reading the vertical vernier.

الأسبـــوع الرابع عشر

عنوان المحاضرة:

Errors in transit work

Errors in transit work:-

Errors in transit work may be instrument, personal or natural.

1)Instrument Errors: - the adjustment, even though carefully made are never exact, otherwise the graduations are not perfect and the centers are not absolutely true.
Errors due to instrument imperfections and / or non adjustment are all systematic and they can eliminated or reduced to a negligible amount by proper method of procedure.

a)Horizontal angles :- when the bubbles of plate levels in non adjustment are centered , the vertical axis inclined , hence the measured horizontal angles are not true angles , also the horizontal axis is inclined to a varying degree depending upon the direction in which the telescope is sighted. For a given direction of sighting, the larger the vertical angle cannot be eliminated by double – sighting, it is important that the vertical axis be made truly vertical either by proper adjustment and use of the plate levels or by telescope level.

If the line of sight is to perpendicular to the horizontal axis, an error in horizontal angle results, if the telescope is plunged between back sight and foresight, the resultant error is doubled. The angular error (E) in the observed direction of any line along with a sight is taken depends both on the angle (e) by which the line of sight departs from the perpendicular to the horizontal axis and the observed vertical angle (a) to the point sighted that can be :-

$\boxed{E = e \sec a \quad (\text{approx...})}$

If the horizontal axis is not perpendicular to the vertical axis, an error in horizontal angle results. The angular error ϴ in the observed direction of any line along which a sight is taken depend both on the angle ($\grave{e}$) by which the horizontal axis departs from the perpendicular to the vertical axis and on vertical angle (a) to the point sighted.

$\Theta = \grave{e} \tan a$ (approx...)

The error, which becomes large for steeply inclined sights , can be eliminated by double sighting.

b) Vertical angles: - Errors occur when:-

i. The line of sight is not parallel to the axis of level tube.
ii. The vertical vernier is not in adjustment.
iii. The plate levels are not in adjustment, methods of reducing or eliminating these errors are mentioned previously.

2) Personal Errors: - personal errors arise from:-

a) The limitations of the human eye in setting up and leveling the transit and making observations.

b) The transit may not be set up exactly over the station.

c) The plate bubbles may not be centered exactly.

d) The vernier may not be set or read precisely.

e) Parallax may exist in focusing.

f) The line of sight may not be directed on point.

All the personal errors are accidental and cannot be eliminated but can be kept within reasonable limits by care in observing.

3) Natural Errors: - source of natural errors are:-

*a)*Settlement of the tripod.

b) Unequal atmospheric refraction.

c) Unequal expansion of parts of the telescope due to temperature changes.

d) Wind, producing vibration of the transit, or making difficult to plumb correctly.

In general, the errors resulting from natural causes are not large enough to effect the measurement of ordinary precision. However, when the transit is set up on thawing ground, large errors are likely to arise from settlement, usually accompanied by horizontal and angular displacement, errors due to adverse atmospheric conditions can usually be rendered negligible by choosing appropriate times for observing.

الأسبـــوع الخامس عشر

عنوان المحاضرة:

First term practical examination

الأسبـــوع السادس عشر

عنوان المحاضرة:

Leveling staves, Types of leveling staves

Leveling staves: -

Is a graduated straight rod made of well-seasoned timber, it is (75) mm wide and (18) mm thick and rectangular in cross-section. The graduations are in meters and decimeters, a brass cap is fitted at the bottom to avoid wear or tear of the staff. The bottom of the staff represents zero reading. All divisions are written in black against white ground, the meter numeral is marked to the right while the decimeter numeral is marked to the left. The graduations are inverted so that they appear upright through the telescope, while taking observations through the telescope the reading should be taken from top to bottom as the portion of the staff under the field of view looks inverted.

Types of leveling staves: -

1) Solid staff: - it is a single piece of (3) m length. Due to the absence of any joint or hinge, better accuracy is obtained in graduations.

2)Folding or hinged staff: - two meters length are joined together by a hinge when required in full length, the two portions are locked together so that the total length become a single, rigid rod.

3) ***Telescope or sap-with staff:*** - it consists of three parts. The upper one is a solid part, while the lower two pieces are hollow from inside, the upper part can slid into the central part. Each length can be pulled up and is held in position by means of brass spring. The total length is (5) m, the upper two parts are each of (1½) m, while the lower part is (2) m length.

4) Invar precision leveling staff: - it is used for a very precise leveling. An invar graduated band is mounted on the wooden staff, tightly fastened at the lower end by a spring at the upper end so that expansion or contraction of the staff has no effect on the invar band.

5) ***Target staff: -*** it consists of a sliding target over a graduated staff. When the line of sight is quite far off. It is very inconvenient to take observations properly, the level man direct the staff man to raise or lower the target till it is bisected by the line of sight, then the staff man observes the reading. A vernier is attached along the sliding target to take precise observations.

الأسبـــوع السابـــع عشر

عنوان المحاضرة:

Leveling Instrument .Type of Leveling Instrument

Leveling instruments: -

The leveling instruments can be divided into three main groups:

1) Dumpy levels   2)Tilting levels   3)Automatic levels

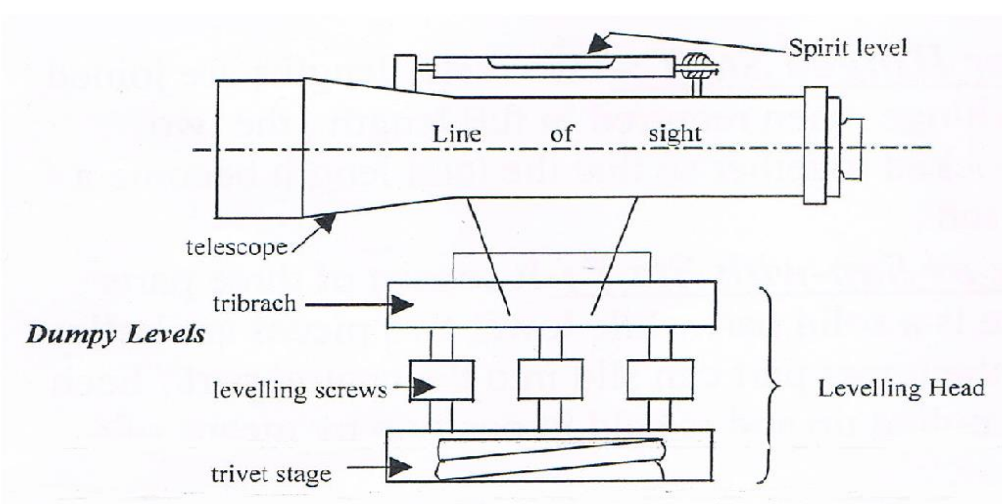
1) ***Dumpy levels: -*** it comprises the following parts as shown below in the figure

a) Trivet stage b) Leveling screw c) Tribrack d) Telescope

e) Spirit level

In dumpy levels the telescope is mounted on a vertical spindle which is free to rotate within the tribrach.

The principle axis is known as the line of sight or line of collimation.



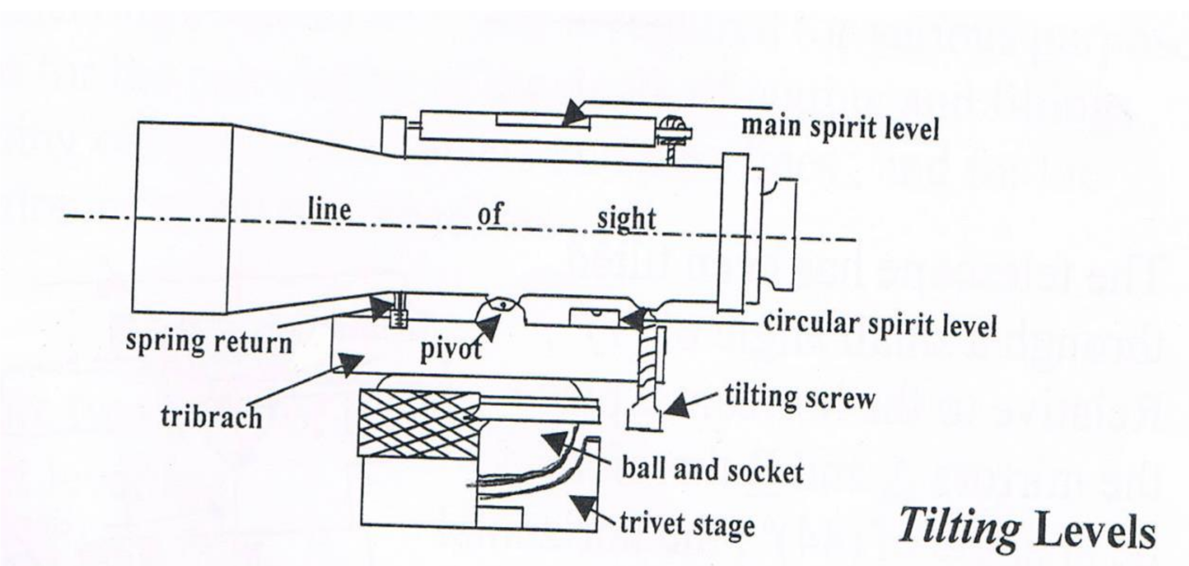
2) Tilting levels: - it comprises the following parts: -

a) The leveling head: - is made up of the same three components as the dumpy level (trivet stage, leveling screw, and tribrach).

b) Spirit level

c) The telescope: - the telescope is not rigidly attached to the tribrach but is supported by a central pivot, it is therefore capable of small amount of vertical movement, this movement gives the instrument a tremendous advantage over a dumpy level.

The vertical movement is imported to the telescope by a tilting screw passing through the tribrach at the eyepiece end of the telescope. A spring loaded return mounted on the tribrach at the objective end of the telescope works, with the tilting screw to elevate or depress the telescope.



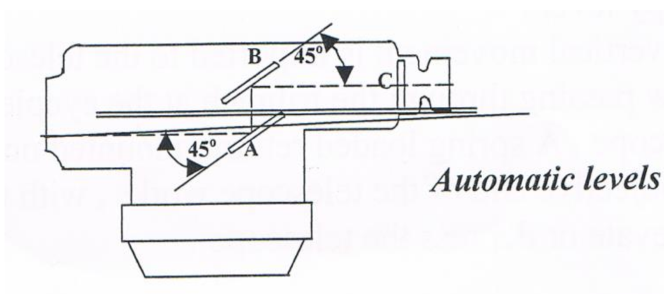
**What is the essential difference between a dumpy and tilting level?

The essential difference between the levels is that the telescope of the dumpy level is rigidly attached to the vertical spindle and can be rotated only in azimuth (i.e. horizontally), while the telescope of the tilting level is supported on a central pivot and can therefore be move both in azimuth and altitude (i.e. vertically).

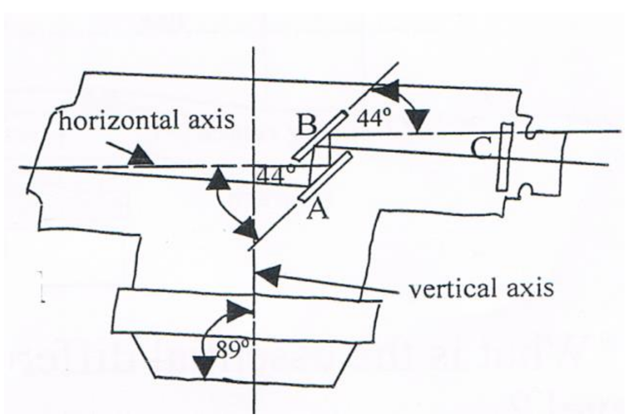
3) Automatic levels: - in the automatic levels the line of sight is leveled automatically (within certain limits) by means of an optical compensator suspended like a pendulum which is inserted into the path of the rays through the telescope.

Basic Principle of the Compensator

In telescopes in which two mirrors have been placed at (45°) to the telescope axis.
The horizontal ray of light entering the objective glass through the optical center will be reflected at (90°) from mirror (A) into mirror (B) where it will once again be reflected at (90°) to pass through the center of the diaphragm (C).
Using this system the compensator (mirror A) must be placed exactly midway between the objective glass and diaphragm.



The telescope has been tilted through a small angle of (1°). Relative to the horizontal plane the mirror A and B therefore lie at angle of (44°), the horizontal ray of light (solid line) entering through the optical center of the objective glass strike mirror A, is reflected to strike mirror B and is reflected from it at an angle of (44°), that is diverging from the original ray (shown dashed) by (1°), it is no longer passes through the center of the diaphragm.



<u>الأسبــوع الثامن عشر</u>

عنوان المحاضرة:

Leveling , Type of leveling .simple leveling, Compound Leveling or Differential Leveling,

<u>Leveling:-</u>

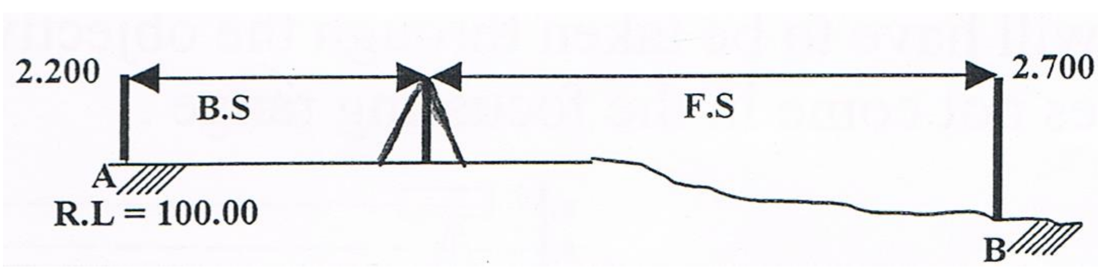
Leveling may be defined as an operation for the measurement of difference in the elevations between points or for the determination of the elevation of certain points above some given plane or surface known as <u>datum surface</u>, generally is taken as the Mean Sea Level (MSL). <u>Leveling</u> is a type of surveying which is carried out for finding <u>difference in heights</u>. In engineering projects leveling is required for various purposes such as for the calculation of the depth of cutting and fillings, for setting out grades for sewers and pipe lines, and for the estimation of reservoir capacities, etc…

<u>Types of leveling: -</u>

There are two types of leveling: -
i) Direct leveling
ii) Indirect leveling

<u>i)Direct leveling:</u> - can be classified into the following categories:-

1)Simple leveling: - This method enables us to determine the difference in level between two points by placing the instrument approximately midway between the two stations, while taking the readings, the bubble remain in the centre as usual.



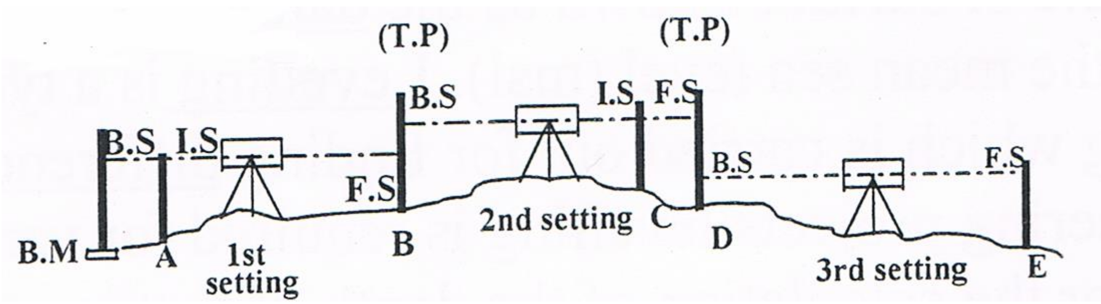
Staff reading at A = 2.200

Height of collimation = 100 + 2.200 = 102.200

Staff reading at B = 2.700

R.L of B = 102.200 – 2700 = 99.500

2) Compound Leveling or Differential Leveling:- This involves the same process as above but in addition, a number of setting of the instrument are required to find out the difference in leveling between two points in the case where a single setting cannot serve the purpose, this may be either due to a large distance involved or due to some intervening obstructions. Each setting is linked with the previous setting by keeping a common point on which observation are taking from both the setting, such point called change point should be a firm, well-defined point as the work has to be carried forward with respect to this point.



3) Flying Leveling: - The reduced levels of some important points is found by a few selected setting of the instrument to get a rough idea of the nature of the ground.

الأسبــوع التاسع عشر

عنوان المحاضرة:

Reciprocal Leveling

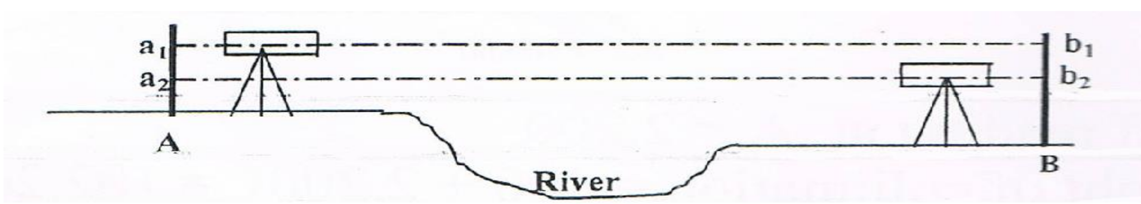
4) Reciprocal Leveling: - When two stations are situated far apart and it is not possible to set up the instrument between these stations because of a river or deep valley and also not possible to equalize the back sight and the fore sight, the process of reciprocal leveling is adopted.

The method avoids applying any correction due curvature, refraction and imperfect line of collimation.

The procedure followed is as follows: -

1) The instrument is set up near A and temporary adjustments are made again.

2) The staff reading are observed at A and B, the observation at A will have to be taken through the objective as otherwise it does not come in the focusing range.



3) The instrument is shifted near B and staff reading are taken at B and A.

instrument at A	instrument at B
Staff reading at A = a**1**	staff reading at A = a**2**
Staff reading at B = b**1**	staff reading at B = b**2**

The true difference (d) between A and B: -

d = 1/2 (b1 - a1) + (b2 - a2)

R.L of B = R.L of A ± d

Where d is rise or fall as the case may be.

الأسبـــوع العشرون

عنوان المحاضرة:

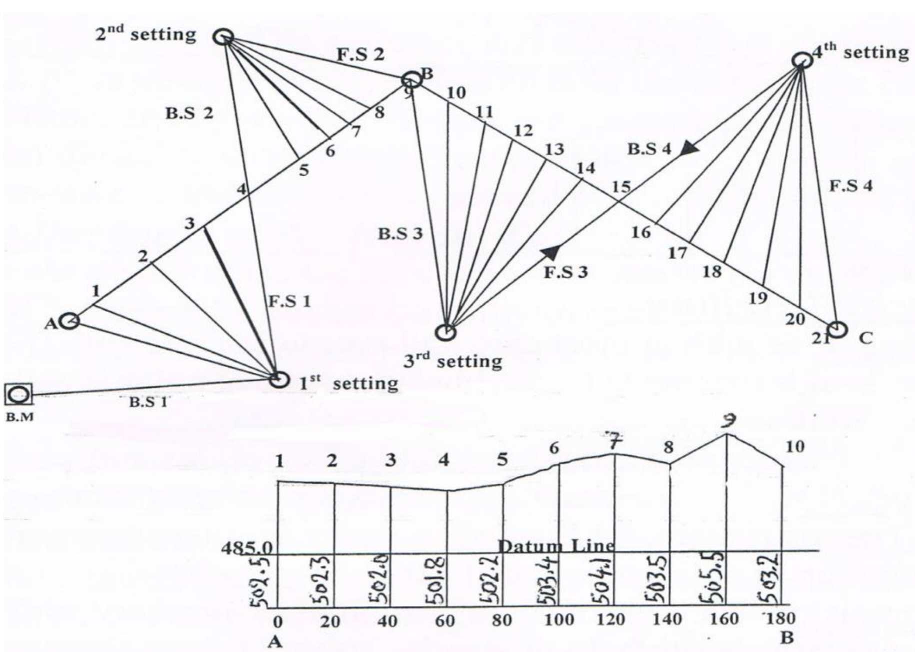
Longitudinal or Profile Leveling

5) ***Longitudinal or Profile Leveling:-***

Longitudinal leveling is an operation of doing a leveling job to determine the profile of a ground surface along a given line, this is generally the center line of proposed alignment, such as rail way, highway, canal, pipeline. The proposed alignment a single or series of straight line connected by curves, the points are selected by uniform interval.

Field work:-

The center line of the proposed alignment is marked on the ground. Points at equal intervals, say 20 m or 30 m are measured. The instrument is set up at some suitable position so as to command the maximum number of points, when it is found necessary to shift the instrument because it is no longer possible to further observation due to larger distances or intervening observations, a suitable change point is selected that the back sight distance is equal to the fore sight distance.



It may be noted that the instrument is always set away from the center line and successive setting are done on either side of the center line.

Plotting a Profile: -

Horizontal distances are marked on horizontal line. A datum line is selected, the R.L of the points are written against the points. Then the points lying on the profile are plotted with respect to the datum. The vertical scale is kept quite large as compared the horizontal scale. Normally the horizontal scale is kept 1 in 1000 to 2000 and the vertical scale 1 in 100 to 200.

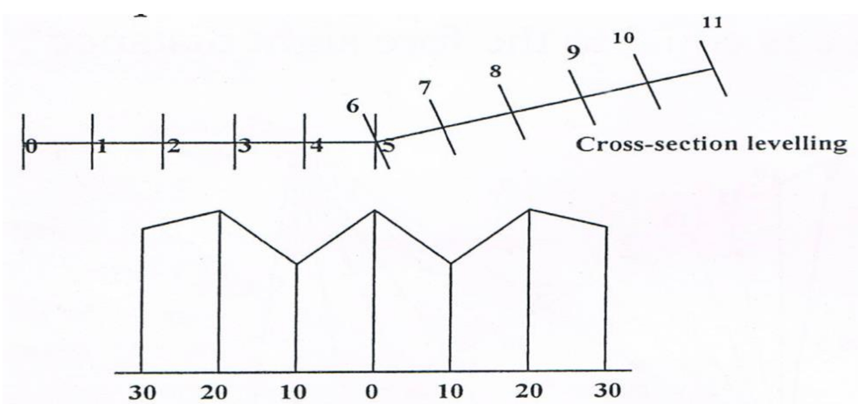
<u>الأسبــوع الحادى و العشرون</u>

عنوان المحاضرة:

Cross-section Leveling

<u>Cross-section Leveling: -</u>

In longitudinal section the center line is taken, but when a wider belt is required for some engineering projects, e.g, high way, rail way …etc, profile leveling alone does not serve the purpose. In such cross-sections are selected at uniform intervals of (30) m or (50) m, the width of the cross-section depends on the purpose. Cross-sections are taken simultaneously along with longitudinal leveling and are serially numbered from the beginning; one or more points are selected on either side of the center line.



<u>7)Precise Leveling: -</u>

Leveling done with at most care and precaution and with most refined instruments to establish the control points is called precise leveling.

A systematic procedure of taking field observations and recording is adapted to minimize and eliminate all source of error. The instruments are fitted with parallel-plate micrometer to observe up to any accuracy of 1/100 of a cm.

Invar precision leveling staff are used in place of ordinary staff, they kept vertical with help of stands, and spirit levels are used to check their verticality.

<u>الأسبـــوع الثاني و العشرون</u>

عنوان المحاضرة:

Adjustment of the Dumpy Level, Adjustment of the Cross-Wire Ring To make the horizontal cross-wire lie in a plane perpendicular to the vertical axis

Adjustment of the Dumpy Level: -

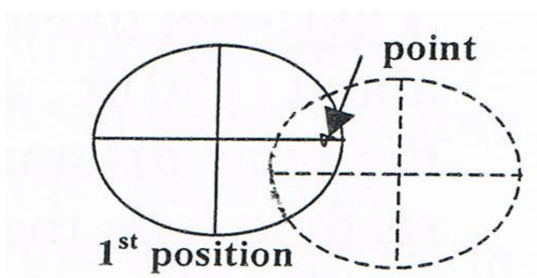
1)Adjustment of the Cross-Wire Ring: -

To make the horizontal cross-wire lie in a plane perpendicular to the vertical axis.

Test:-

Set up the level and sight some definite fixed point in the field of view, the bubble does not to be exactly centered for this test and so, if necessary, the telescope may be moved up or down with the foot screws, and to the right or to left on its vertical axis, until one end of the horizontal cross-wire is fixed on the point.

Then turn the telescope about the vertical axis until the other end of the cross-wire has reached the second position shown by dashed lines.



2nd position

if the relation cross-wire (horizontal) perpendicular to the vertical axis exists, the point will appear to move along and will remain on the horizontal cross-wire, if not as shown in figure above, the point will appear to move off the cross-wire.

Adjustment: -

The adjustment is made by slightly loosening both pairs of capstan screws which hold the cross-wire ring in position and by turning the ring with pressure of the fingers, when the adjustment is complete the point will remain on the cross-wire as the telescope is moved slowly from side to side.

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الأسبـــوع الثالث و العشرون

عنوان المحاضرة:

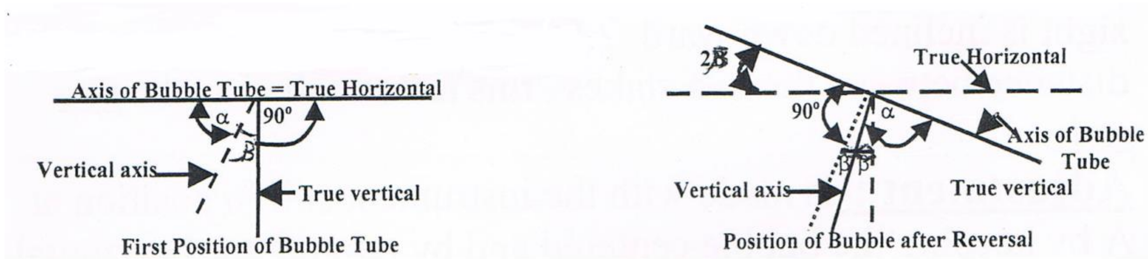
Adjustment of the Bubble Tube , Adjustment of Line of Sight

2) Adjustment of the Bubble Tube: -

To make the axis of the bubble tube perpendicular to the vertical axis.

Test: -

Center the bubble carefully over both pairs of leveling screws and bring it exactly to center one pair, then turn the telescope about end for end over the same pair of screws, if the correct relation exists, the bubble will remain centered, if not, it move away from the center and the amount of movement indicates double error of adjustment, it noticed that it is evident that the assumed error in the relationship is represented by the angle ß and that after reversal the inclination of the axis of the bubble tube is equal to 2 ß**.**



Adjustment: -

The bubble is adjusted by the capstan screw back half way with amount represented by the angle ß, and then the bubble brought to the center with foot screws by moving the vertical axis through the same angle ß, if the adjustment is complete, the bubble will remain centered both before and after reversed, thus proving that its axis is perpendicular to the vertical axis.

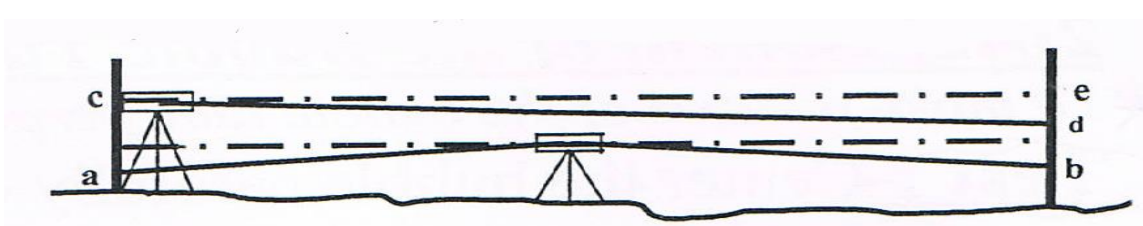
3)Adjustment of Line of Sight: -

To make the line of sight parallel with axis of the bubble tube.

Test: -

Set the level up and drive on stake A at a distance of about (150) ft, and another stake B in the opposite direction at the same distance. Take a rod reading a on the first stake and a reading b on the other stake, obviously the difference in reading (b – a) is the true difference in elevation between the two stakes, regardless of any error in the instrument, further more C & R effects both readings equally.

Now; set the instrument up near stake A so that the eyepiece will swing very close i.e. within (1/2) inch of the rod, and read the rod by observing through the objective lens which represent the reading at C, this reading will be without error, if the true difference in elevation (b – a) is added to the reading C, the sum will be the reading e which may be called the computed correct reading on stake B that would be obtained if the line of sight were truly horizontal.



Thus assume reading as follows: -

a = 2.04 b = 5.16 c = 4.73

Then b – a = 3.12 ft

Since b is larger than a, the stake B is lower than A and therefore: c+ (b – a) = e

or 4.73 + 3.12 = 7.85 ft the computed correct reading on stake B.

Then the reading d = 7.81 is taken, it is evident that the line of sight is inclined downward 7.85 – 7.81 = 0.04 ft for the distance between the two stakes, this test is called the **peg test.**

Adjustment: -

Adjustment is made with the instrument is still in position at A by keeping the bubble centered and by moving the horizontal cross-wire until it apparently cuts the rod at the correct reading, in the case assumed, the cross-wire will be moved until the rod reading is 7.85 ft.

الأسبـــوع الرابع و العشرون

عنوان المحاضرة:

Sensitivity of Bubble Tub

Sensitivity of Bubble Tub: -

It is defined as the number of seconds of central angle subtended by one division, usually (2) mm of the bubble tube.

Test: -

Set up the instrument, sight the rod, and with bubble in any noted position in the tube, carefully read the cross-wire on the rod which is at a taped distance D from the instrument, then by manipulating the foot screws, move the bubble an exact number, say five of divisions and read the rod again, the difference between the rod reading divided by 5 will be the intercept (I) in feet.

Example: -

The intercept found by reading a level rod twice, at a distance of (300) ft, the bubble being moved five divisions between reading is (0.15) ft, the length of the bubble division is (2) mm. find the sensitivity V.

Solution:

I = intercept

D= 300 ft and i= 0.03 ft

Tan V = 0.03/300 = 0.0001 ⟵ tan V = i / d

V= 20″ very nearly

الأسبــوع الخامس و العشرون

عنوان المحاضرة:

Errors in differential leveling and their correction

Errors in differential leveling: -

1) Non adjustment of the instrument: -

a) Adjustment of cross-wire ring
b) Adjustment of the bubble tube
c) Adjustment of line of sight

errors due failure of the line of sight to be parallel with axis of the bubble tube are eliminated by equalizing back sight and fore sight distances, this can be done by pacing or estimation, for more careful work the sight distances are measured by stadia.

2) The Bubble not centered: -

If the bubble is not centered when the rod is read, an accidental error in the reading results, the magnitude of the error depends on the sensitiveness of the bubble tube, the source of error is minimized by carefully watching the bubble at the time the rod reading is taken.

3) Incorrect reading of the rod: -

This source of error is due to the fact that the eye is not able to judge exactly where the horizontal cross-wire apparently cuts the rod, it is an accidental error and its magnitude depends on the distance to the rod, the qualities of the telescope, parallax, the weather conditions … etc, the error should not exceed (0.005) ft at the distance of (300) ft.

If the weather conditions are adverse, the length of the sight is shortened correspondingly, when long sight are necessary, as when crossing a wide river, reciprocal leveling procedure may be employed.

4) The rod not plumb: -

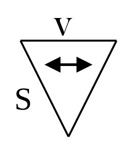
When the rod is not plumb when a reading is taken, a positive systematic errors results, the magnitude depends on the size of the rod reading, it is greater near the top than the bottom of the rod.

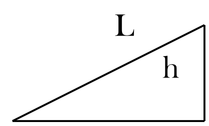
Example

Calculate the error in an observed rod reading of 10.00 ft if the rod is out of plumb by 6 inch at that height?

Solution:

Error = $V^2/ 2S = (0.5)^2/2*10 = + 0.01$ ft





C**S**= $h^2/2L$

This source of error is minimized by carefully plumbing the rod for all reading, this is done by the rod man standing squarely behind the rod and balancing it between the finger tips of both hands. In all times when readings are taken near the top of the rod, it is a good practice for the rod man to (wave the rod) slowly toward and away from the level and the level man to make the minimum reading.

5) Parallax: - parallax is a source of accidental error, it is eliminate by focusing the eyepiece on the cross-wires and this should be made before beginning any work with the level.

6) Curvature of the earth and atmospheric refraction: -

It is noticed in a long run, if a considerable numbers of unbalanced sights are taken i.e. fore sights longer than back sights or vice versa, an appreciable error may result which makes always the rod reading in differential leveling too great.

7) Incorrect length of the rod: -

Causes a systematic error, but this again is rendered accidental in the process of leveling provided the difference in elevations are not great, if a line of level is carried from bottom to the top of the hill (or vice versa), a serious error will result if the rod has even a small error in its length.

The error is to be minimized by testing the length of the rod from time to time by comparing its length with a steel tape.

Example:-

The preliminary differences of elevation between two points was found by differential leveling to be +404.68 ft, it was then discovered that the (12) ft rod was actually (12.025) ft long with the error distributed uniformly over the length of the rod, what is the correct difference of elevations?

Solution: -

Correction = (404.68/ 12)*(0.025) = +o.84 ft

The revised difference of elevation = +405.52 ft

Sometimes an error is caused by improper joining of the section of the rod, if this condition is detected the rod should be discarded. This condition will also result if, in extending the rod, the rod man is not careful to extend the rod its full length.

8) Setting of the instrument: -

In some soil the level may settle in the interval of time between a back sight and a fore sight reading, for this condition extra precaution are taken to ensure the stability of the level.

9) Poor turning points: -

A well designed turning point will not serve it purpose when set in loose sand or swampy soil.

10) Heat waves: -

Heat waves which affect the precision of the rod reading, this is an accidental error to be minimized by limiting the length of the sights.

11) Wind: - Wind may vibrate the instrument and make it difficult to keep the bubble centered and to read the rod correctly.

الأسبـــوع السادس و العشرون

عنوان المحاضرة:

Electromagnetic Distance Measurement (EDM) , Basic concept of measurement

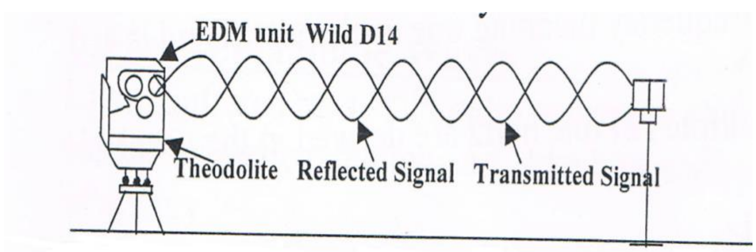
Electromagnetic Distance Measurement (EDM): -

Modern developments in electronics have now made possible the measurement of distance using the electromagnetic signals. The measurement is accomplished in seconds with very high degree of accuracy.

The instruments were first introduced during the 1950´s and at present there are some sixty different instruments in the market.

Basic concept of measurement: -

The basic concept is simple. An EDM instrument capable of transmitting an electromagnetic signal is set up over a survey station at one end of a survey line.



The signal is directed to a reflector, where it is reflected or re-transmitted back to the transmitter.

The transit time of the double journey is measured by the transmitter, and since the speed of light is accurately known, the distance is calculated from the formula:
Distance between stations (m) = velocity of signal (m/s) × transit time(s)

$D = V \times T$

The electromagnetic signal which is transmitted in the form of radio waves, infrared light, visible light or laser beam all of which have different properties, although they travel at the same speed.

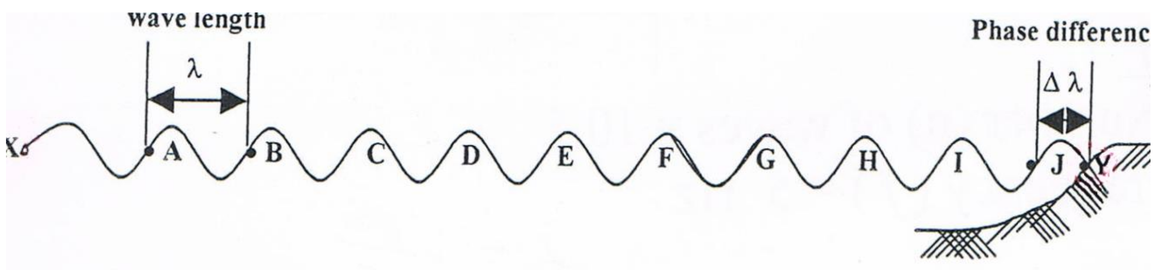
Properties of the signal: -

A direct analogy can be made between the movement of electromagnetic wave and that of waves caused by dropping a stone into a pool of water.

a)Water waves : -

i) wave length and phase difference: -

A stone has been dropped at X, and the pattern of wave is seeing emanating from that point to the bank at point Y.



The distance between any two similar points is obviously the length of the wave, so XA=XB= wave length.

The waves are said to be in phase when a complete number of them occur between starting and finishing point.

In case of incomplete portion of wave finishing on y, is said phase difference of (0.5) wave length.

ii) Frequency: -

If the wave take (2.1) seconds to travel from X to Y, its frequency (*f)* of occurrence is

f =10.5 times per 2.1 seconds

That means that wave makes 5 cycles per second.

The SI unit of frequency meaning one cycle per second is 1 hertz.

The various multiples of one hertz are derived in the usual manner:

1 hertz = 1 Hz

10^3 hertz = 1 kilo hertz = 1 KHz

10^6 hertz= 1 mega hertz= 1 MHz
10^9 hertz = 1 giga hertz =1 GHz

If the wave length λ measures 0.5 meter, the distance XY is found thus

Distance XY= wave length λ × frequency (f)

= 0.5 ×10.5 = 5.25 m

The velocity (V) of travel of this wave is therefore:

Velocity (V) = distance (D) ÷ time (t)

= 5.25 ÷ 2.1

= 2.5 meter per second

= 2.5 m/s

$$\boxed{\begin{array}{c} D = V \times T \\ V = D / T \end{array}}$$

Example: -

A stone dropped into a pool at X generates a wave which travels through the water with frequency of (5) Hz, and at a velocity of (2.5) m/s. A count the waves shows that (10.5) waves occur by the time the wave reaches the bank Y, calculate the distance XY.

Solution:

Number of waves (n) = 10.5
Frequency (f) = 5 Hz
Time = 10.5/5=2.1 s
Now, velocity (V) = 2.5 m/s (given)
Distance D_{xy} = velocity (V)× time (t)
2.5×2.1 = 5.25 m
From above D= V × t
= V × n / f

This method and the derived formula are important in EDM and indeed from the basis of the system.

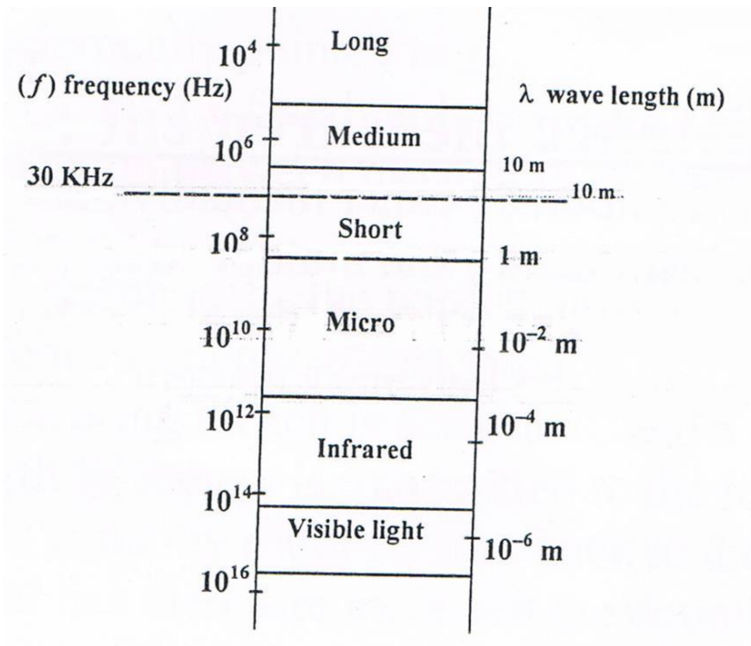
b) Electromagnetic waves: -

Light, infrared rays and raid waves are all forms of electromagnetic radiation, and like heat and sound are forms of energy.

i)wave length and frequency: -

Electromagnetic waves behave in much the same manner as wave in water.

The figure below show part of electromagnetic spectrum, the wave length of the various bands vary from 10000 m long waves to 0.001 mm visible light waves, the corresponding frequencies begin 30 KHz and 30×10 KHz respectively
Only a narrow band of these waves can be used to measure distance to the standard of accuracy required in surveying.



The table below shows the list of few of EDM instruments in common use and shows the frequency and approximate wave length of the measuring signal

Instrument	Frequency	Approximate wave length
Wild M.D 60		
Fine measurement	144.8483 MHz	2 m
Coarse measurement	14.97733 MHz	20m
Kern DM 500		
Fine measurement	14.9854 MHz	20 m
Coarse measurement	149.854 KHz	2000 m

Wild D.I 3		
Fine measurement	7.4927 MHz	40 m
Coarse measurement	74.927 KHz	4000 m
Geodimeter 6	30 MHz	10 m
Tellurometer CA 1000	(19 – 25) MHz	(16 – 12) m

These ranges of frequencies are not suitable for direct transmission through the atmosphere by EDM instruments, because the waves tend to scatter and suffer from interference.

<u>ii) Velocity: -</u>

The velocity V through the atmosphere is therefore = 299708.0 km/s approximately

If at the end of measurement the values of temperature, pressure, and humidity differ from the standard values, correction must be made to the measurement of line.

<u>Principle of distance measurement: -</u>

The value of (n/f) is computed either manually or automatically by the instrument, and multiplied by the standardized speed of the signal through the atmosphere. The result is the slope length of the line measurement.

الأسبـــوع السابع و العشرون

عنوان المحاضرة:

EDM systems , Microwave system (long range) .

EDM systems:-

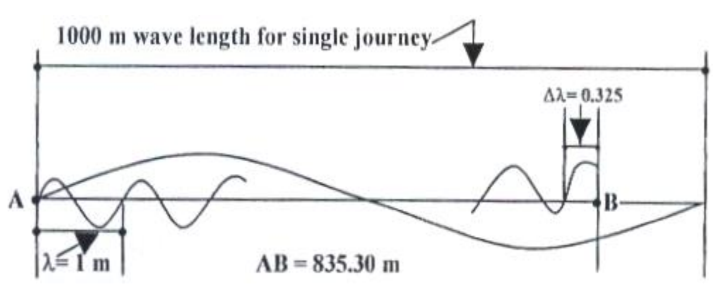
Can be divided in to two classes:-
1) Microwave system (long range) .
2) Electro-optical system (medium and short range).

1)Microwave system :-

This group of EDM instruments uses microwaves to measure distance from (20) m to a maximum of (150) km, with an accuracy of about (3-4) mm per km.
An instrument typical of this class is the wild M.D 60 which operates on frequencies of about (15) MHz

Field work:-

In figure below, the length of line AB is known to be 835.30 m, in order to check the length using microwave methods , two instrument are required , the instrument may be identical as with the wild M.D 60 or may be different and not interchangeable as with the tellurometer CA 1000 .



i) One instrument called the master is set at A and pointed in the direction of B. The second instrument called the remote is set at B and reciprocally pointed to A .

The instruments are fitted with powerful radiotelephones, so communication between the parties is always possible over long distances.

Using the telephones, the parties make contact and tune their respective instruments to each other.

ii) The measuring switch is activated, and a microwave of wavelength W meters is transmitted to the remote instrument where it is instantly retransmitted back to the master instrument, the signal has therefore traversed the double distance AB, the fraction part of the waves is called the phase difference that is accurately measured by the instrument.

wave length λ	Equivalent λ for single journey	Phase difference
w_1 2 m	1 m	0.325
w_2 20 m	10 m	0.532
w_3 200 m	100 m	0.353
w_4 2000 m	1000 m	0.835

iii) The wave length is then changed to w2, and the phase difference is again measured. This process is repeated a certain number of times on different wave lengths , the number depending on the make of instrument and length of the line **,** these wave lengths changes are made automatically with the Wild M.D 60 , the whole measuring procedure taking approximately thirty seconds (30). A typical set of wave lengths and the phase difference obtained on the measurement of line AB as shown in table above.

Measurement of atmospheric conditions:-

At the finish of the measuring procedure, measurements of the prevailing temperature, pressure and humidity must be made, since they may differ from the standard. The measurements are made using wet and dry bulb thermometers and a barometer.

The measurements are made at both ends of the line, if the line is long, there is no guarantee that the measured conditions actually prevail along the complete length, may be responsible for errors of about (2) mm per km.

Calculation of the distance:-

l) Number (n) of wave lengths.
For example, using wave length $w_4(\lambda = 2000m)$ where the phase difference $\Delta\lambda = 0.835$, the number of complete wave lengths is found as follows:-

$\Delta\lambda = 0.835$

But $w_4 = 1000\ w_1$

$\therefore\ \Delta\lambda \times 1000 = 1000 \times 0.835 \quad = 835$

The total number of wave lengths (complete and partial) is derived thus

$1000 \times\ \Delta\lambda(0.835) = 835.0$

$100 * \Delta\lambda(0.353) = 35.3$

$10 * \Delta\lambda(0.532) = 5.32$

$1 * \Delta\lambda(0.325) = 0.325$

$\therefore$ No of wave lengths = 835.325

2) Transit time (t)

$t = n / f$

$$= \frac{(835.325)}{149.8483000 \times 10^6} \text{ second}$$

$t = 5.574471 \times 10^{-6}$ second

3) Distance <u>AB</u>

$2 * D(AB) = V \times t \qquad V = 299708$ km/s

$2 * AB = 5.574471 \times 10^{-6} \times 299708 \times 1000)m$

$= 1670.650$ m

$\therefore AB = 835.356$ m

Atmospheric correction for (3) C° temperature and (800) mm Hg is – 0.009m

$\therefore$ AB = 835.356

– 0.009

835.347 m

AB = 835.35 m

2) *Electro-optical system:-*

The instrument employed in tills system divided in to two classes,

l) The instruments which use visible light from the medium-range class.

2) While those using infrared light from the short-range class .

The measuring signals are carried on a narrow, highly focused beam of light which has to be directed optically to the distant target by means of telescope.

The short-range EDM units may be mounted on the telescope of a theodolite.

The medium-range Geodimeter 710 model is a purpose built instrument which combines an electronic digital theodolite with an EDM unit as a calculator. All measurements are presented in digital form.

Field work:-

1) The transmitter is placed at one end of the line being measured and is accurately aligned by means of the telescope on to a corner-cube reflector at the other end.

The important property of the reflector is that it reflects light back along any line on an exactly parallel course , one corner-cube reflector is effective for ranges up to about (600)m , for longer length , nine prisms is required .

2) The signal is transmitted on a known frequency to the reflector , from which it is returned to the instrument and the phase difference is measured .

3) The frequency is changed either manually or automatically by the instrument and the measuring procedure repeated , enabling the number of wave length to be counted and the slope distance to be calculated.

4) Measurement of the prevailing atmospheric conditions are made , and corrections are applied to the slope length by means of the nomograph supplied with some instruments .

5) The vertical angle between instrument and reflector is measured in the usual manner.

The generation of the signal and the method used to count the number of wave lengths differs with the two groups of instrument.

<u>الأسبـــوع الثامن و العشرون</u>

عنوان المحاضرة:

Medium-range instruments

<u>a)Medium-range instruments :-</u>

The Geodimeter (6) the medium-range group of instrument

<u>1) Signal generation:-</u>

The light source is a normal (5V) lamp powered by battery which is used for measuring lengths up to (5) km long in day light or (15) km long during darkness.

<u>2) Wave length count:-</u>

The three wave lengths used for the measurement are:-

w_1= 5.000000m

w_2= 4.987532m

w_3= 4.761904m

These lengths are chosen such that

$400w_1 = 401\ w_2 = 2000m$

and $20\ w_1 = 21\ w_3 = \quad 100m$

Using the same line <u>AB</u> = 835.300 m, the phase difference resulting from measurement on the wave lengths w_1, w_2 and w_3 are

$\Delta w_1 = 0.300$

$\Delta w_2 = 2.382$

$\Delta w_3 = 1.967$

Distance <u>AB</u> $= nw_1 + \Delta w_1$ _ _ _ _ _ _ _ _ _ _ _ _ (1)

$= nw_2 + \Delta w_3$ _ _ _ _ _ _ _ _ _ _ _ _ (2)

$= nw_3 + \Delta w_3$ _ _ _ _ _ _ _ _ _ _ _ _ (1)

From (1) and (2),

$n(w_1 - w_2) = \Delta w_2 - \Delta w_1$

And since $400\ w_1 = 401\ w_2$

400

$$w_2 = \left(\frac{\quad}{401}\right) w_1$$

$$n\left(w_1 - \frac{400}{401}\right) w_1 = 2.382 - 0.300$$

$$\frac{n\, w_1}{401} = 2.082$$

$$n\, w_1 = 834.9$$
$$= 835$$

This value will be repeated every <u>2000</u> m

From (1) and (3) ,

$$n\,(w_1 - w_3) = \mathbf{\Delta} w_3 - \mathbf{\Delta} w_1$$

And since $20\, w_1 = 21\, w_3$

$$w_3 = \left(\frac{20}{21}\right) w_1$$

$$n\left(w_1 - \frac{20}{21} w_1\right) = 1.967 - 0.300$$

$$\frac{n\, w_1}{21} = 1.667$$

$$nw_1 = 35$$

This value will be repeated every <u>100</u> m .

The total number of wave length is therefore 8354

$\mathbf{\Delta} w_1 = 835.30$

الأسبـــوع التاسع و العشرون

عنوان المحاضرة:

Short- range instruments

b) Short-range instrument:-

i) Signal generation:-

All modern short-range instruments emit an infrared carrier wave generated by gallium arsenid (GaAs) diode. The wavelength is less than 1 micrometer.

The power is supplied by nickel-cadmium dry-cell batteries or by a (12) volt car battery.

The beam is invisible and harmless and will produce correct distance even when it is broken by traffic.

ii) Wave length count:-

All instruments of this class are completely automatic, the number of wave lengths is counted by some electromechanical device and the slope distance is displayed digitally .

Operation of an infrared instrument:-

The kern D.M 502 is a typical instrument of this class , the unit is easily attached to a kern theodolite by means of spring clippower is supplied from a Nicd battery strapped to the tripod and connected to EDM by flexible cable .

The measuring operations are as follows :-

1) Switch on the power.

2) Set the theodolite line of sight on to the distant reflector.

3) Set the function switch to measure.

4) Press the starting button marked measure.

The instrument then measures the phase difference on two different frequencies and the distance compute automatically and displays it digitally, the complete operation take about (15) seconds.

Slope correction:-

The distances obtained by EDM instrument are slope lengths.
The plane length must be calculated.
Plane length (D) = slope length (L) × cosine vertical angle (0)

Or plane length (D) = slope length (L) – $\frac{h_2}{2L}$

h = the difference in elevation between instrument and target.

<u>الأسبـــوع الثلاثـــــــــون</u>

عنوان المحاضرة:

second term practical examination