STUDY THE ACCURACY OF MICRONETWORK FOR PRECIS ENGINEERING PROJECTS

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الخلاصة :

ساعد التطور التكنلوجي السريع في اجهزة الكمبيوترفي التغلب على مشكلة تحسين وتقييم دقة الشبكات التي تستخدم للاغراض الهندسيه الدقيقه .في هذا البحث تم تقييم ومقارنة دقة مواقع النقاط عن طريق المساحة التي يغطيها الخطا(الذي يكون على شكل قطع ناقص) لكل نقطه من النقاط في حالتي استخدام خط قاعده واحد واستخدام خطي قاعده للشبكه المستخدمه في مراقبة سد حديثه. وقد تم التوصل الى ان استخدام خطي قاعده يعتبر من الامور التي تساعد في تحسين دقة الشبكات ومعالجة الضعف الموجود في الشكل الهندسي للشبكه.

Abstract

The rapid development in computer technology helped to overcome the problem of accuracy improvement and accuracy assessment of high precision network. In this paper a study was conducted to evaluate and compare the accuracy of position through the area of error ellipses in case of using single base line and two base lines for geodetic monitoring network of Haditha Dam .

The present paper purpose is to conclude that the use of additional baseline is an important part in the improvement of micronetwork accuracy and treats the weakness in the geometrical figures used.

1- Introduction:

Over the last decade, an interest has grown among the civil engineering and building professions in monitoring the movement of different types of structures both during and after completion of construction. There are many reasons why a structure may need to be monitored for movement. For examples, It is well known that dam walls change shape with varying water pressure, that the foundations of large buildings are effected by changes in ground conditions, and that landslips sometimes occur on embankments and cuttings. For all of these deformations surveys can be used to measure the amount by which a structure moves both vertical and horizontal over regular time intervals. The purpose of the micronetwork that using for precise engineering projects are to ascertain if movement is taking place and to assess whether a structure is stable and safe. In addition, movement may be analyzed to assess whether its is due to some daily seasonal or other factors, and most importantly it may be used to predict the future behavior of structures. Priori to any surveying project engineers have to estimate and determine the accuracy of surveying. This paper deals with the solution of accuracy improvement for high precision network. In this paper above problems are treated by means of the variance covariance matrix. This matrix can be computed for any network unrespectable of whether or not actual observations have been carried out. The variance covariance matrix is used for plotting station error ellipses. The geodetic monitoring network of Haditha Dam (shown in figure 1-1) is used for this purpose.

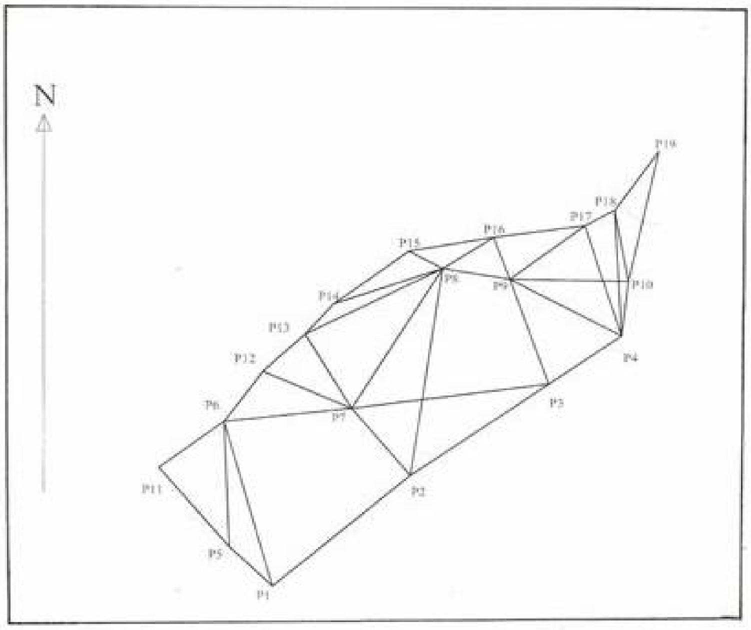

Figure (1) Geodetic Network of Haditha Dam

Scale 1:50000

2. Control surveys

Engineering surveys are usually based on horizontal and vertical control networks which consist a fixed point called control station. Series of control stations forming a network can be used for the production of site plans [1].

Methods of determining the horizontal positions or rectangular coordinates of control point include traversing triangulation and trilateration. In addition, horizontal control can be extended using intersection and resection.[1]

A triangulation network consists of a series of single or overlapping triangles as shown in Fig. (2)in which the points of each triangle forming control station. Position is determined by measuring all the angles in the network and by measuring the length of one or more baselines such as AB or HJ in Fig. (2).

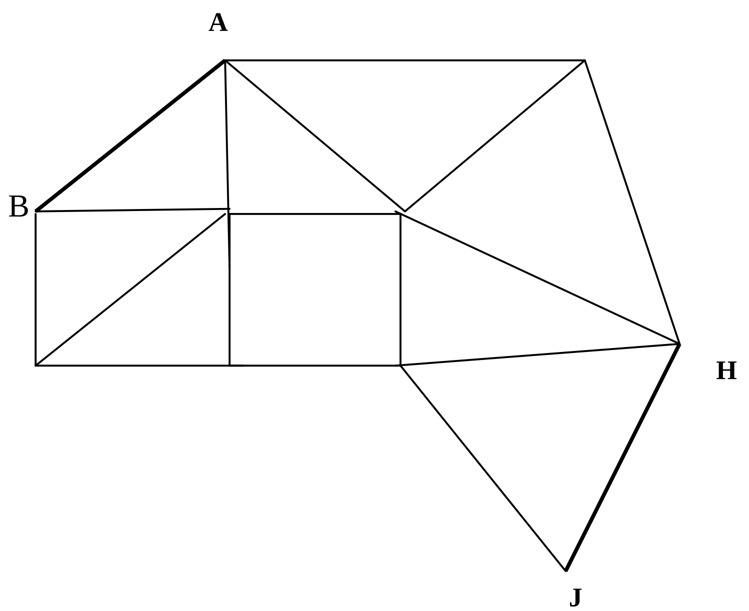

Figure (2) triangulation network

Starting at the baseline and application of the sine rule in each triangle throughout the network enables the length of all triangle sides to be calculated. These lengths when combined with the measured angles enable coordinates of each stations to be computed.

A trilateration network also takes the form of series of single or overlapping triangles but in this case position is determined by measuring all the distances in the network instead of all the angles. For all trilateration surveys the measurement of the meteorological conditions at all times is vital and atmospheric corrections must be rigorously applied to all EDM measurements. For station coordinates calculation the measured distances are combined with angle values derived form the side lengths of each triangle.

Because of high precision and accuracy of modern EDM equipments, traversing, triangulation and trilateration can all be used as methods of establishing horizontal control. On construction sites, combined networks are used horizontal control is required to provide reference points for control extension, for monitoring, and for precise engineering work.

Two types of horizontal control network are used for deformation monitoring: absolute and relative networks. An absolute network is a network in which one or more points are considered to be stable so that a reference datum is provided against which coordinate changes can be assessed. One of the more difficult problems with an absolute network is to identify and confirm the stability of fixed reference points. A relative network is one type in which all surveyed points are assumed to be moving and has no stable datum.

3. Base Lines:

Because of the angular measurement errors the accuracy of the computed lengths will decrease as the distance from the baselines increase.. On extensive triangulation the accuracy is maintained by measuring additional baselines. The required frequency of the bases is dependent on the strength of figures. The use of the EDM's eliminates the need for elaborate baseline preparations [2]

4.Least Square Adjustment:

Nowadays, network coordinates are often calculated using methods based on least squares. A least squares adjustment, however, accounts for all angles and distances measured in a network, making full use of all the redundancy in a network, performs a simultaneous adjustment of field data and calculation of coordinates. In other words, least squares adjustment will produce a single solution no matter how the original data is collected and processed. In addition to computing the best adjustment, Least squares is also capable of providing a complete analysis of survey including details of the positional accuracy of each coordinated station [3]. This information can be used at planning stage to ensure that a survey meets its specification .The development of of least squares adjustment theory are based on the variance law for independent observations .In the observation equation method the adjusted observation are expressed as linear function of suitable parameters as [4]:

A X = L+V ------------------(1)

Where:

X = column vector of parameters

A = matrix of coefficient

L = column vector of correlated observation

V=column vector of residual .

The application of least squares adjustment in plane coordinate surveys includes formulation and linearization of the equation (distance and angles) encountered in the adjustment of plane coordinates by the method of indirect observations (variation of coordinates), least squares position adjustment for a typical procedure employed in plane coordinate surveying as shown in Fig. (3) below.

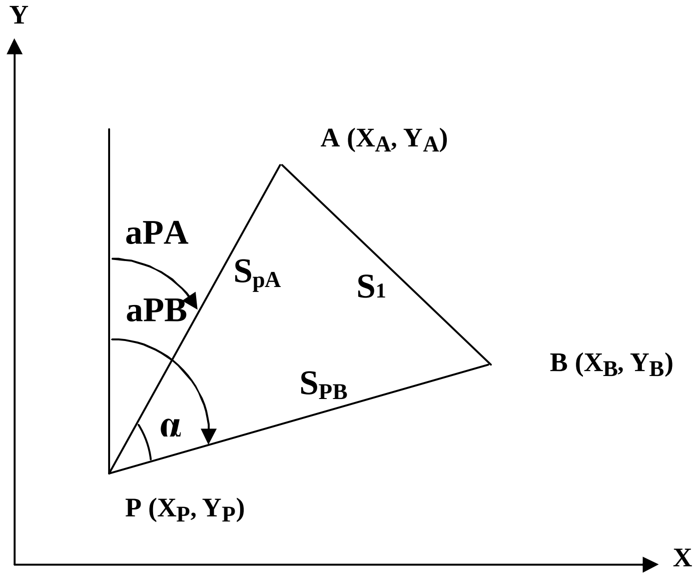

Figure (3): Interrelation of Horizontal Angles, Distances, Azimuths and(x,y) Coordinates in Plane Surveying

The observation of the measured angle shown in Fig. (3) can be expressed as [5]:

$$\propto +V\propto = aPB - aPA = \tan^{-1}\left(\frac{XB-XP}{YB-YP}\right) - \tan^{-1}\left(\frac{XA-XP}{YA-YB}\right) \quad (2)$$

Where:

$\propto$: angle from A to B.

$V\propto$: residual of $\propto$.

(XA,YA),(XB,YB),(XP,YP) the plane coordinates of A,B and P.

The non –liner form for equation (2) can be made linear by Taylor's method .

Final linear form of equation (2) is given as follows:[7]

$$v\propto + \propto = \tan^{-1}\left(\frac{(X_{PO}-X_{Po})}{(Y_{Bo}-Y_{po})} - \tan^{-1}\left(\frac{X_{Ao}-X_{po}}{Y_{Ao}-Y_{Po}}\right)\right) +$$

$$\int^{I}\left[\frac{Cos(A_{pBo})}{S_{PBo}}\Delta XB - \frac{Sin(a_{PBo})}{S_{PBo}}\Delta YB + \frac{Cos(a_{PAo})}{S_{pAo}}\Delta XA - \frac{Sin(a_{PAo})}{S_{PAO}}\Delta YA + \left[\left(\frac{Cos(a_{PAo})}{S_{PAo}}\right) - \frac{Cos(a_{PBo})}{S_{PBo}}\right]\right]$$

$$\Delta XP + \left[\frac{Sin(a_{PBo})}{S_{PBo}} - \frac{Sin(a_{PBo})}{S_{PAo}}\right]\Delta YP$$

Where:

(X_{AO}, Y_{AO}),(X_{BO}, Y_{BO}),(X_{PO}, Y_{PO}): approximate coordinates of A,B and P.

a_{PAO}, a_{PBO}: approximate azimuth of PA and P B.

S_{PAO} , S_{PBO}: approximate distance of AP and BP.

$\int$: a coefficient to convert radians to grade.

ΔXA, ΔYA, ΔXB , ΔYB, ΔXP, ΔYP the correction of approximate coordinates.

The observation equation of a measured distance SPA as shown in figure (3) is

$$S_{PA} + V_{PA} = \sqrt{(XA - XP)^2 + (YA - YP)^2} \quad \text{--------------------} \qquad (4)$$

where :
(XA,YA),(XP,YP) : plane coordinates of A,P

The linear form of Equation (4) is given as follows [7]

$$S_{PA} = S_{PAO} + V_{SPA} = Sin\ (a_{PAo})\ \Delta XA + Cos(a_{PAo})\ \Delta YA - Sin\ (a_{PAo})\ \Delta XP - Cos(a_{PAo})\ \Delta YP \text{---------------}(5)$$

Where:

S_{PAO} : approximate distance of line AP.

a_{PAo} :approximate azimuth of line AP.

Δ XA, Δ YA , Δ XP, Δ YP :the correction of coordinates of A,P

V_{SPA}: *residual of distance AP.*

For geodetic monitoring network of Haditha Dam shown in figure (1) measurements are achieved by state commission on survey. For angular measuring used T3 theodolite instrument and for distance measuring used DI5S instrument. The side p_1p_5 is a baseline and by using the approximate coordinates, (63) observation equation can be formed for angles and (39) observation equations can be formed for distances after substituting in equation (3) and (5) respectively. When use another baseline (p_4p_{10}) and by using approximate coordinates, (63) observation equation can be formed for angles and (38) observation equation can be formed for distances after substituting in equations (3) and (5) respectively.

By using matrix algebra shown in equation (1), the least squares solution for finding the vector X is obtained by applying the condition V^TV= minimum.

The above condition, if applied in equation (1) gives so called normal equations [5].

$(A^TA)\,X + A^T\,L = O$ ----------- (6)

Where A^TA: a square symmetrical matrix (covariance matrix)

X: column vector of parameters .

A: matrix coefficient.

L: column vector of correlated observation.

The above procedure of adjustment and solution were performed on a microcomputer using the software LOTUS.

5. Error Ellipses:

Besides providing the critical information regarding each points precision, a major advantage of error ellipses is that they afford on excellent means of making visual comparison of point precisions.

The size, shape, and orientation of station error ellipses depend upon the precision of the various observations and on the geometric character of the network. It is, therefore, possible to estimate in advance of an actual survey the priori precisions of observations of a proposed network. Using simulated observational data, a least squares adjustment can be performed for this purposed survey, and station error ellipses can be plotted. The network geometry can then be varied into differing on configurations, least squares adjustments performed on these differing configurations, and error ellipses plotted for each of the variations. Besides varying geometry, a priori precisions can also be varied in accordance with precisions that would be possible for the available survey equipment.

By experimentation, it is possible to select the optimum network and equipment combination needed to achieve a given precision for any survey to derive the semi major axis and semi minor axis and the orientation of the semi major axis the covariance matrix $(A^T A)^{-1}$ or $(A^T{}_P A)^{-1}$ [8] and [9].

$$(A^T A)^{-1} = \begin{pmatrix} Qxx & Qxy & Qxz & \cdots \\ Qxy & Qyy & Qyz & \cdots \\ \vdots & \vdots & \vdots & \\ Qxz & Qyz & Qzz & \cdots \\ \vdots & \vdots & \vdots & \end{pmatrix}$$

$$\tan 2t = \frac{2Qxy}{Qyy - Qxx} \quad \text{-------------} \quad (7)$$

Where t is the orientation of semi- major axis.

$$Quu = \tfrac{1}{2}\,[Qyy+Qxx+K] \quad \text{---------- --} \quad (8)$$

$$Qvv = \tfrac{1}{2}\,[Qyy+Qxx-K] \quad \text{-------------} \quad (9)$$

$$K = \sqrt{(Qyy - Qxx)^2 + 4(Qxy)^2} \quad \text{----------} \quad (10)$$

Where:

Quu: numerical values when multiplied by $(S_O{}^2)$ gives variances along (u) axis, The square root of this variance gives the semi -major axis of the error ellipses.[9] and [10].

Q_{vv}: numerical values when multiplied by $(S_O{}^2)$ gives variances along (v) axis, the square root of this variance gives the semi major axis of the error ellipse (see figure (4)).

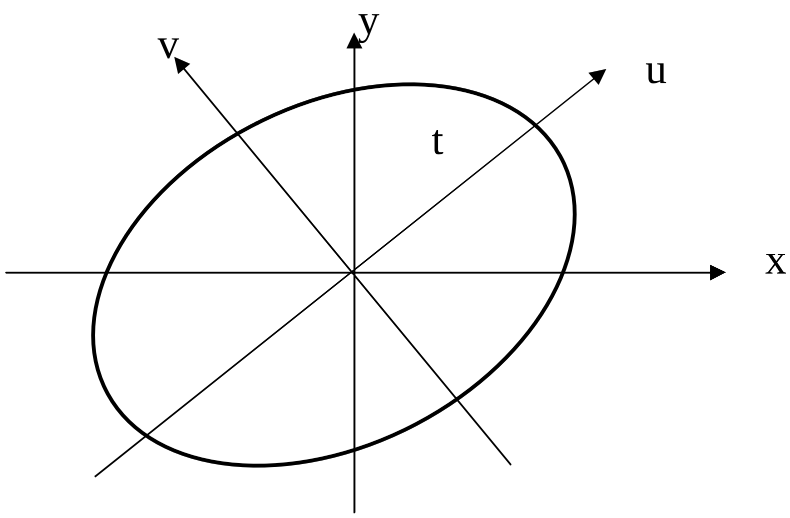

Figure (4) Error Ellipses

As shown in figures (5) and (6) the amount of errors in the positions of points 16,17,18 and 19 is increasing as we go away from the baseline p_1p_5 because of the weakness in the geometrical figures used. In order to decrease the errors in the network and to keep the amount of errors to be acceptable we use p_4p_{10} baseline in the opposite side of original baseline .This was clearly shown in table (1) and see the figures (7) and (8). The mean of area of network error ellipses when use p_1p_5 as base line equal to 156.33 mm^2 , but when use p_1p_5 and P_4P_{10} as a base the amount will be 97.56 mm^2 as illustrated in table (2),

From above ,it can be conclude that the use of another base line in the opposite side of original base line is an essential requirement to improve the micro network accuracy.

Table (1) Difference in Semi-major axis and Semi-minor axis Between Network when use (p_1p_5) Base line and when use (p_1p_5) and p_4p_{10} Baselines

Position	network when p_1p_5 baseline		Network when (p_1 p_5) and (p_4 p_{10}) baselines	
	a	b	a	b
2	0.011	0.002	0.011	0.002
3	0.024	0.002	0.015	0.002
4	0.010	0.003	—	—
6	0.012	0.001	0.013	0.001
7	0.012	0.001	0.011	0.001
8	0.021	0.001	0.012	0.001
9	0.022	0.003	0.011	0.002
10	0.027	0.003	—	—
11	0.009	0.002	0.008	0.003
12	0.013	0.002	0.015	0.002
13	0.015	0.002	0.013	0.002
14	0.018	0.005	0.016	0.003
15	0.021	0.003	0.020	0.002
16	0.024	0.003	0.013	0.002
17	0.027	0.003	0.019	0.003
18	0.029	0.003	0.017	0.003
19	0.034	0.003	0.018	0.003

Table (2) Difference in Area of Ellipses Between Network when(p_1p_5) Base Line and when use (p_1p_5) and p_4p_{10} Baselines. *

position	area when p_1p_5 baseline(mm^2)	area when (p_1 p_5) and (p_4 p_{10}) baselines(mm^2)
2	69.11	69.11
3	150.79	94.24
4	94.24	_
6	37.70	40.84
7	37.70	34.56
8	37.70	37.70
9	207.24	69.08
10	254.34	_
11	56.52	75.36
12	81.64	94.20
13	94.20	81.64
14	282.60	150.72
15	179.91	125.61
16	226.188	81.64
17	254.34	178.98
18	273.18	160.14
19	320.28	169.56

* Area=($\pi \times a \times b$) (see table (1))

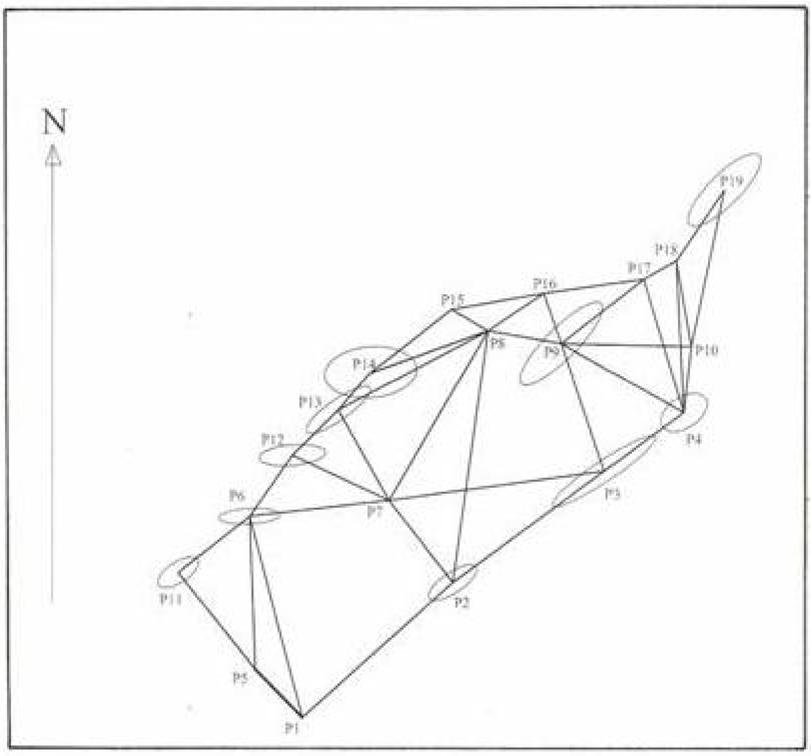

Fig. (5): Error Ellipses for some points

Network Scale 1:50000

Ellipse Scale 1:1

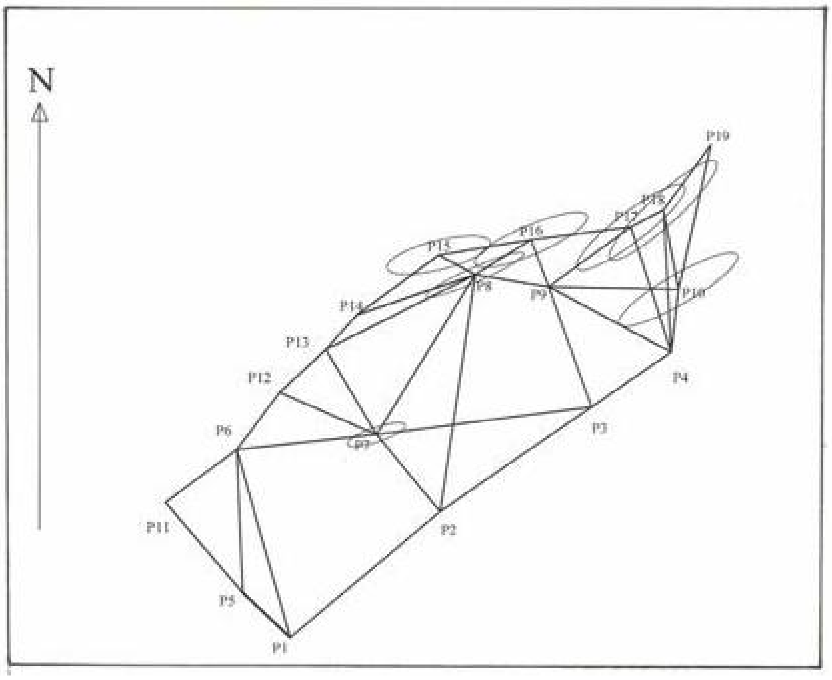

Fig. (6): Error Ellipses for other points

Network Scale 1:50000

Ellipse Scale 1:1

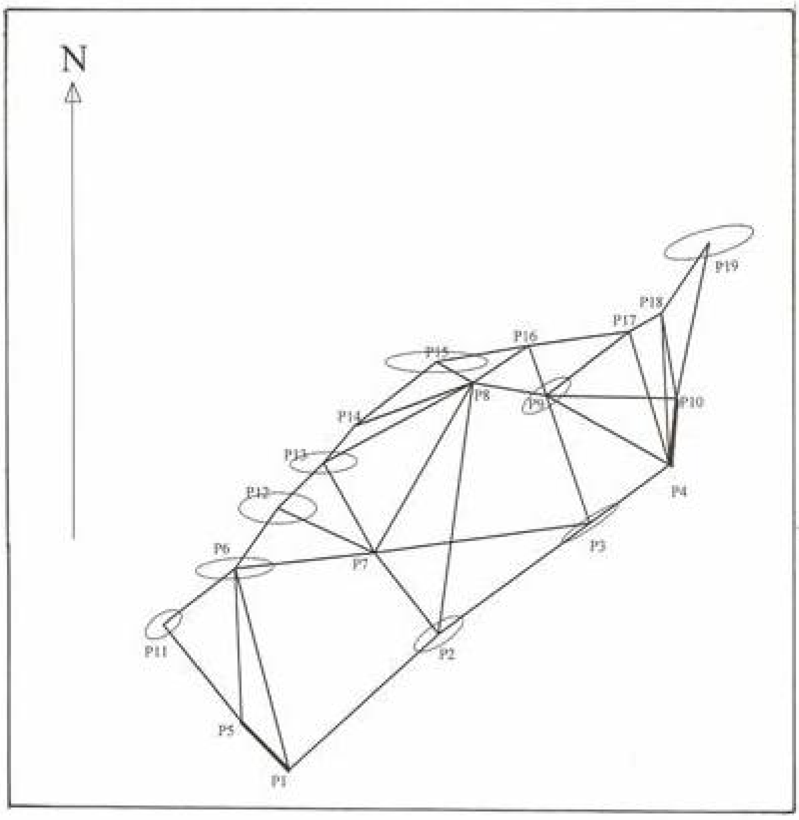

Fig. (7): Error Ellipses for some points

Network Scale 1:50000

Ellipse Scale 1:1

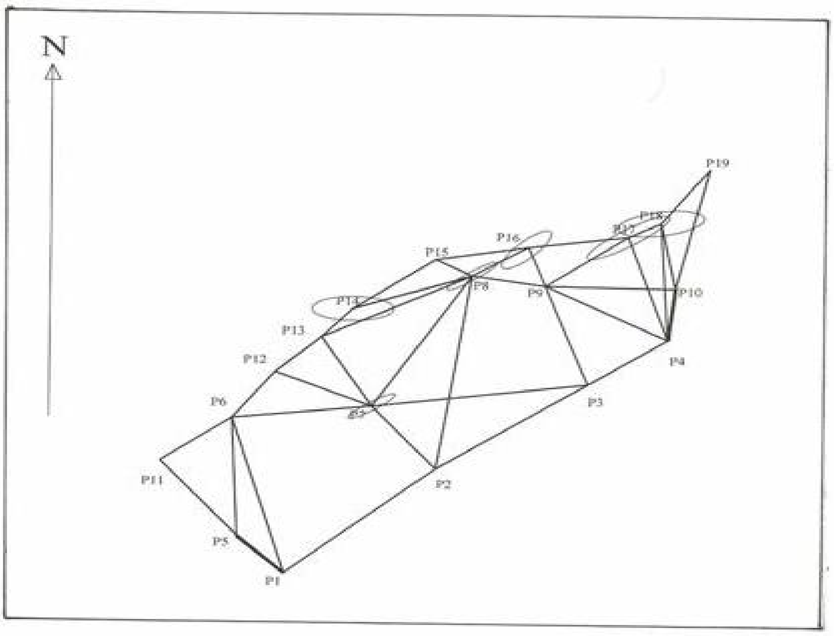

Fig. (8): Error Ellipses for other points

Network Scale 1:50000

Ellipse Scale 1:1

Conclusion:

The use and study of accuracy have been obtained through the variance covariance method for analyzing micronetwork accuracy, which has been successfully applied to Haditha Dam. The following conclutions are drawn:

1-To decrease the network errors caused by a weakness in geometrical figures and away from the first baseline, using another baseline is an essential requirement.

2- Major advantage of error ellipses is that they afford an excellent means of making visual comparison of point precisions as well as it is possible to select the optimum network.

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