Feasibility of use of single frequency GPS receivers in establishing and augmenting control networks

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The main aim of this research paper is to study the feasibility of using GPS single frequency receivers in establishing small geodetic networks as an alternative or addition to the traditional EDM/ Theodolite instruments. A practical field work was carried out by establishing a test network including 5 stations in two observation missions: The first mission includes continuous GPS observations for 24 hours on each baseline, using two single frequency receivers (10 days of observations). The second mission includes EDM/ Theodolite observations for all baselines, using a total station. The first network was adjusted at first by using the variation of coordinates method three times namely: Commercial Software adjustment, Vectors GPS network adjustment, and Projected network adjustment. While the second network was adjusted by using the variation of coordinates method three times as: Trilateration, Triangulation, and Hybrid network. Computation in different stages was carried out using personal built-up software developed in the research, based on integration between MATLAB and EXCEL platforms. The comparison analysis was carried out based mainly on the criteria of scale, shape, and orientation according to the network type. Results of analysis have shown that, the average difference between the GPS and the optimum network was 13 PPM, while it was 10 PPM, between the traditional and the optimum network. The results of this research has shown that GPS single frequency receivers, using suitable observation parameters and analysis techniques, can be used for establishing geodetic network of 3rd order, or less, with high reliability and precision.

يهدف البحث إلى دراسة جدوى استعمال مستقبلات GPS الاحادية التردد فى إنشاء الشبكات الجيوديسية كبديل لشبكات المثلثات والتضليع التقليدية أوالاضافة لها. تم إجراء بحث عملى بإنشاء شبكة ذات خمسة نقاط وتم رصدها على مرحلتين: الاولى بقياس فرقى بارصاد GPS لمدة 24 ساعة لكل متجه. والمرحلة الثانية تم فيها قياس مسافات EDM وزوايا باستعمال جهاز المحطة المتكاملة. وتم تصحيح أرصاد المرحلة الاولى ثلاث مرات: الاولى بواسطة برمجيات تجارية والثانية كشبكة متجهات والثالثة كشبكة مستوية على مسقط ميركيتور. واما المرحلة الثانية فقد تم ضبطها على هيئة شبكة مثلثات وشبكة تضليع وشبكة مشتركة. وتم إجراء دراسة مقارنة مبنية على معايير المقياس والشكل والاتجاه تبعاً لنوع الشبكة. وقد اجريت الحسابات فى جميع مراحل البحث بواسطة برمجيات شخصية مطورة خصيصا للبحث باستعمال برامج MATLAB و EXCEL، وعرضت نتائج البحث وأهمها نتائج ايجابية بالنسبة لجدوى استعمال مستقبلات(GPS) أحادية التردد فى إنشاء واستكمال شبكات الربط الجيوديسية.

Keywords: Global Positioning System (GPS), Geodetic network adjustment, Single frequency receiver

1. Introduction

Since the GPS became a world choice for geodetic networks as compared to traditional techniques, all national agencies and surveyors started using GPS for establishing new geodetic networks and intensifying the existing networks by new control points of the same or lower degree. Previous studies recommended using dual frequency receivers in differential static mode to observe geodetic baselines with occupation time of 2-4 hours, which will produce sub centimeter accuracy. Modern technology and new electronic revolution have made it possible for single frequency receivers to give better precision than before. It is expected to obtain sub-meter or centimeter accuracy by using suitable techniques for observation and data analysis, [1].

In this research, a practical study involving using single frequency GPS receiver for establishing a geodetic network with baselines less than 750m was conducted. Such a network can be classified as 3rd degree network. Two sets of observations were performed on this network:

- Traditional lengths and angles observations using total station.
- GPS observations using single frequency receivers.

The aims of this practical study are:
1. Feasibility of establishing geodetic network by using GPS Single frequency receivers.
2. Determination of the optimum observation parameters of using single frequency receivers in geodetic works.
3. Study of the different adjustment methods for both of traditional lengths and angles networks on one side, and GPS networks on the other side, and comparing their internal accuracy in case of using personal built-up software.
4. Comparing observed single frequency GPS baselines with traditional EDM baselines.
5. Using the GPS observations for reorientation of existing traditional networks; in local coordinate systems; to the global UTM coordinate system
6. Investigation of a new technique for merging GPS and traditional network's observations

2. Network observations

A net of five stations was constructed in the vicinity of Faculty of Engineering Alexandria, Egypt. Stations were chosen on building roofs to provide an open sky for GPS observations, fig. 1. Two observation missions were conducted. The first was for GPS data collection using single frequency GPS receivers; (SOKKIA STRATUS), and the second was traditional lengths and angles observations using a total station; SOKKIA POWERSET 3010.

2.1. GPS data collection

GPS observations of the network, fig. 2 were taken in June, 2005. The observations took place for about 24 hours for each baseline. Observed data were transformed from receiver to pocket PC through infrared connection, and then downloaded to a personal computer through link cable.

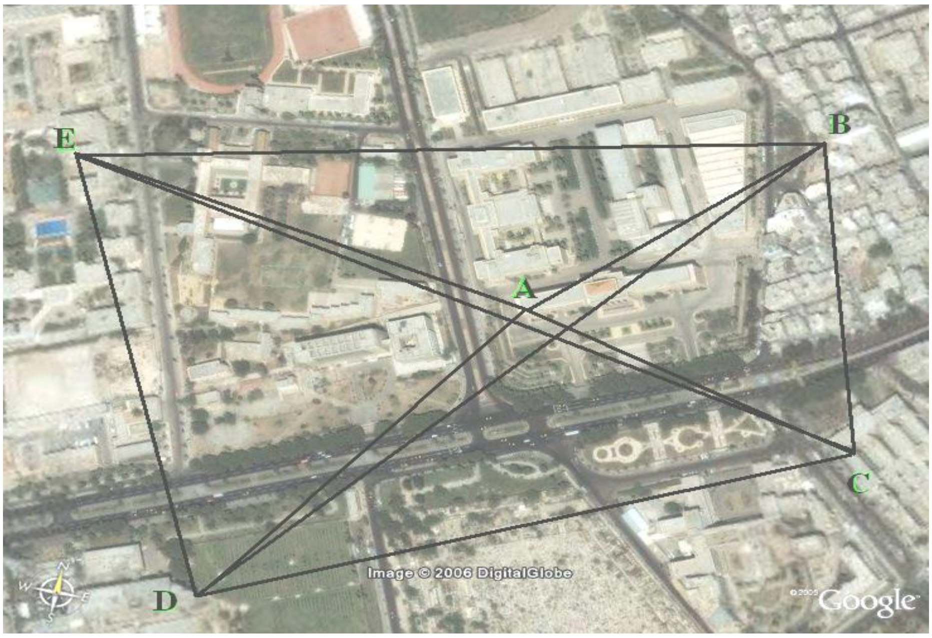


Fig. 1. Study network.

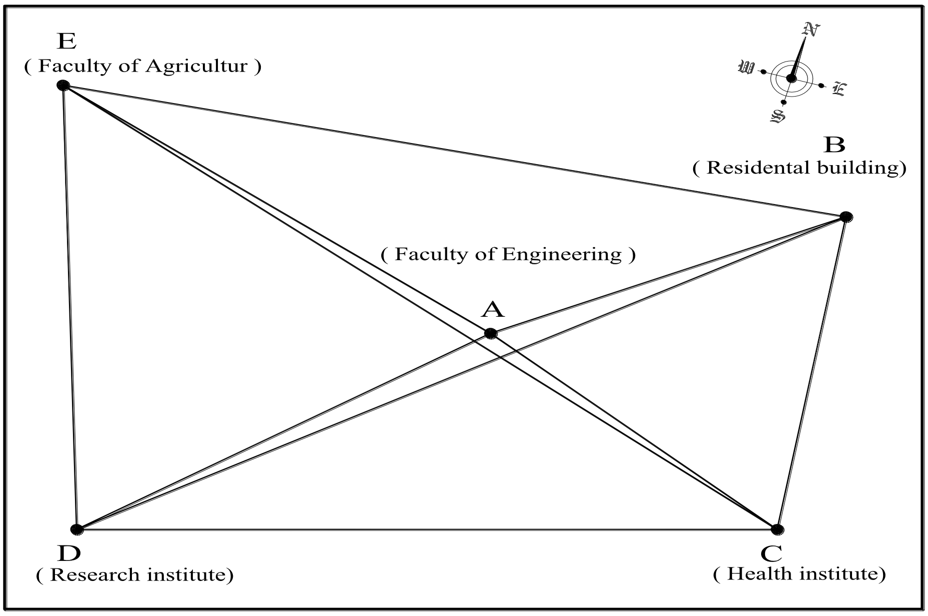


Fig. 2. GPS network baselines.

2.2. Total station data collection

All available baselines slope lengths, vertical angle, and horizontal angles were measured to form the traditional network, fig. 3. Data was downloaded from total station to a personal computer through link cable.

3. Reduction of data and preliminary analysis

3.1. Traditional network

In this section, the required analysis, calculations, and adjustment for the traditional network are outlined. The network consists of five stations and 10 distances ranging from 200-700m. Firstly, the data was reviewed to eliminate blunders. Then a preliminary data analysis was performed to calculate the most probable values of observations, their standard deviations, and the results were used to calculate approximate coordinates for unknown stations.

Secondly, a basic network adjustment was performed. Least squares theory (Variation of coordinates method) was chosen for adjustment, starting with the approximate coordinates. Adjustment matrices were formed by Microsoft Office Excel program, and calculation operations were performed using MATLAB program. The network was adjusted 3 times as follow:

A: Trilateration network adjustment.
B: Triangulation network adjustment.
C: Hybrid network adjustment.

Error analysis was performed based on Variance- Covariance matrices, [2]. The pervious procedure can be outlined in the following flow chart, fig. 4.

3.1.1. Preliminary analysis

The objectives of this analysis stage are:

- Calculation of the most probable values for the measurements.
- Reduction of the measurements to the UTM plane.

Firstly, the closure angular error was corrected at each point, the averages and residuals were computed, and then standard deviation of each angle was calculated. Secondly, the average slope lengths of baselines were calculated where available, standard deviation of each baseline was calculated based on instrument specification as eq. (1). Finally, the horizontal ground distance H was calculated from the measured slope distance *S*, and the average measured vertical angle α, through eq. (2).

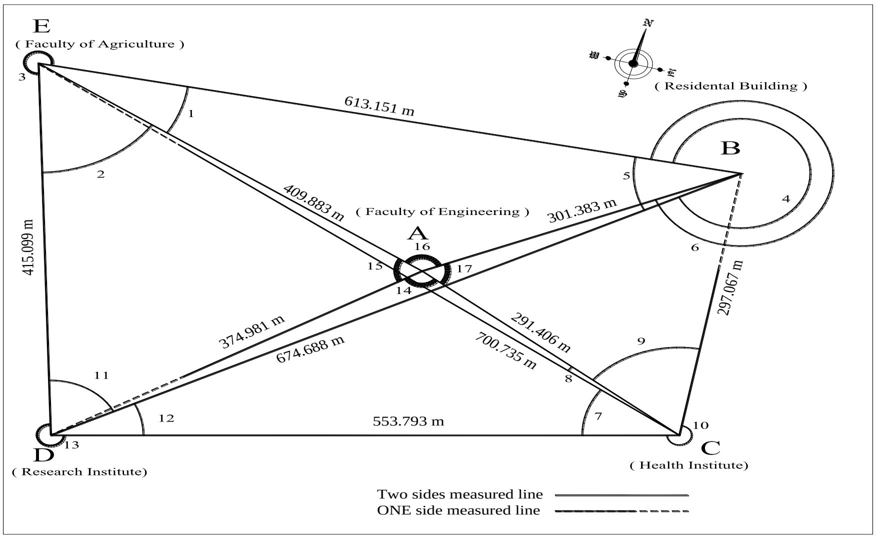


Fig. 3. Traditional network observations.

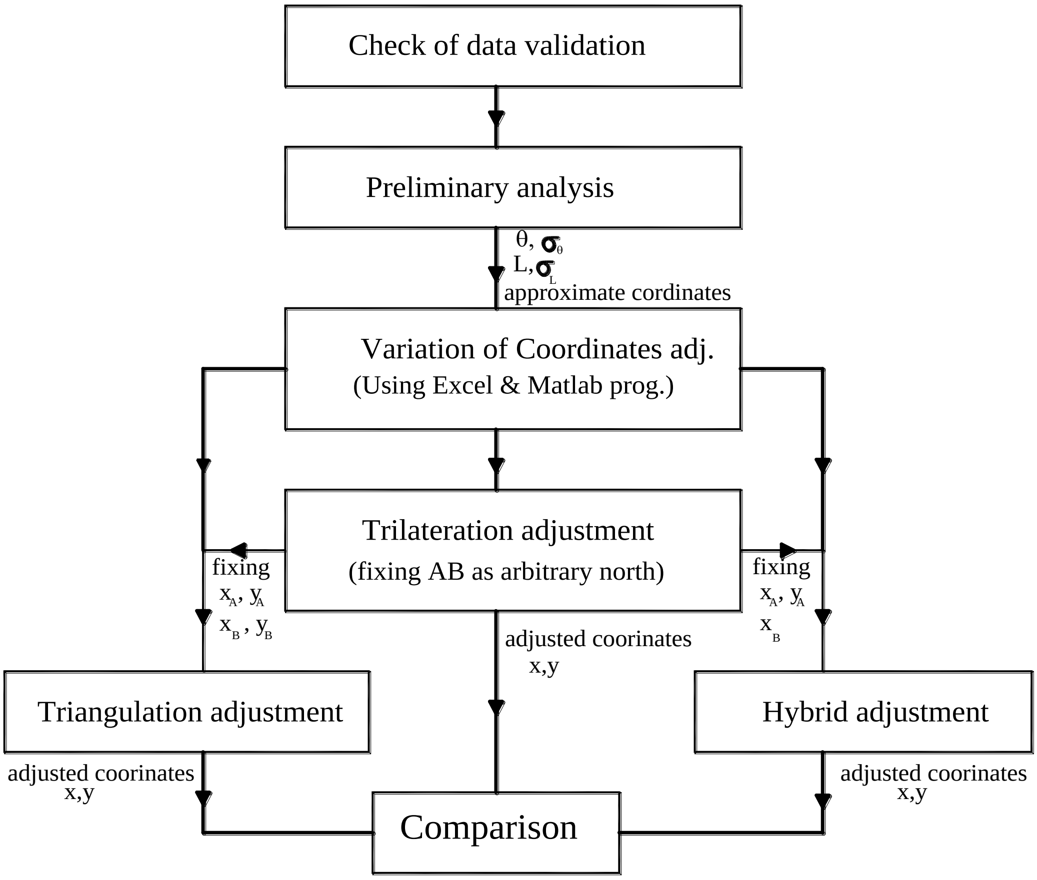


Fig. 4. Traditional network processing and adjustment strategy.

$$\sigma\, mm = 2 \pm (2PPM)\,. \quad (1)$$

$$H\,\text{dist.} = S\,\text{dist.} \cdot \cos\alpha\,. \quad (2)$$

Since the main aim of this study is to compare traditional terrestrial networks with GPS geocentric networks, a transformation should be made to convert both networks into one common system. Both traditional and GPS networks will be transformed to UTM system. To project the traditional measurements on the UTM system, two types of corrections should be applied: The first is the Spherical excess correction for angles, which can be neglected for networks with baselines less than 12 km [4], and the second is the distance correction by applying two factors:

- UTM projection scale factor K.
- Elevation reduction factor H_f .

Projection scale factor: Is the ratio between the projected distance between two points and their corresponding geodesic distance. The line scale factor is the average of the two end points scale factors K, which can be calculated using eq. (3). For short baselines, the two end points scale factors can be considered equal, so the line scale factor can be calculated using the same eq. [3].

$$K = K_0 \left(1 + \frac{\left(\Delta\lambda^2\right)\cdot\left(\cos^2\phi\right)}{2}\right). \quad (3)$$

Where:

K: The scale factor

K_o: Scale factor on the central meridian (for UTM K_o = 0.9996)

$\Delta\lambda$: Difference between point meridian and the central meridian of zone

ϕ: Point longitude

Elevation reduction factor: is the ratio between the ellipsoidal distance, and the corresponding ground horizontal distance. It can be computed using eq. (4). This elevation factor is a function of the average height above the reference ellipsoid, [3].

$$H_f = \left(\frac{R}{R+H+N}\right), \quad (4)$$

where:

H_f: Elevation reduction factor

R: Mean radius of the ellipsoid

H: Mean height of the line above the mean sea level

N: Mean geoidal height of the line above the ellipsoid

Knowing the ellipsoidal height h instead of H, the eq. (4) can be written as:

$$H_f = \left(\frac{R}{R+N}\right). \quad (5)$$

The above two factors were merged in one new factor C_f, which can be given as:

$$C_f = (K)(H_f)\,. \quad (6)$$

The UTM grid distance can be computed from measured ground distance as:

$$UTM\,grid\,distance\,. \\ = C_f \cdot Ground\,horizontal\,distance\,. \quad (7)$$

Since the study network baselines were less than 1 km long, the factor C_f was calculated at the central point A (Faculty of Engineering) of the network, and was considered as a constant value for all network.

The fixed known coordinates of station a [8] are:

ϕ = 31° 12' 21.46656" N λ =29° 55' 26.34023" E
h = 62.185 m $R \approx$ 6370 km

For UTM projection station (A) located in zone 35, which has a central meridian λ = 27°, by substitution into eqs. (3 – 6) the following factors were obtained:

$\Delta\lambda$ = 2° 55' 26.34023" E K = 1.00055224
H_f = 0.99999024 C_f = 1.00054248

By applying eqs. (1, 2, and 7) to the measured sloped distances, the horizontal distances and the reduced UTM distances were calculated.

3.2. Basic network adjustment

The collected conventional measurements; was distributed into 3 groups to form the traditional trilateration, triangulation, and hybrid networks observations [4]. Since there was only one known station A (Faculty of

Engineering) in the network and there is no defined azimuth. A local plane coordinates system was assumed taking station A coordinates (1000E, 1000N), and AB direction as an arbitrary north (with azimuth = 0°); [5, 6]. The approximate coordinates for unknown points B, C, D, E were computed using lengths of baselines AB, AC, AD, AE and angles $\theta 14$, $\theta 15$, $\theta 16$, $\theta 17$; fig. 3.

The adjustment procedure was carried out according to the following sequence:

A- Trilateration adjustment: the observations in this case were 10 baseline lengths and one azimuth. All adjusted coordinates and their error parameters were calculated. Then, the adjusted line AB was fixed for triangulation and hybrid adjustment.

B- Triangulation adjustment: line AB was fixed from trilateration adjustment. The observations in this case were 1 baseline length and 17 angles, all adjusted coordinates and their error parameters were calculated.

C- Hybrid adjustment: only AB start and direction was fixed from trilateration adjustment. The observations in this case were 10 baseline lengths, 17 angles and one azimuth, all adjusted coordinates and their error parameters were calculated.

The adjustment process was carried out according to the following model, [6]:

Weight matrix of observations W

$$W = \frac{1}{\sigma^2} . \tag{8}$$

Observation equations:

$$W \cdot A \cdot X = W \cdot K + W \cdot V . \tag{9}$$

The corrections matrix X:

$$X = (A^T \cdot W \cdot A)^{-1} \cdot A^T \cdot W \cdot K . \tag{10}$$

The Variance-Covariance matrix of the unknowns

$$C = (A^T \cdot W \cdot A)^{-1} . \tag{11}$$

The final adjusted coordinates of network points, standard deviations are shown, table 1.

3.3. GPS Network adjustment

In this section, the Spectrum Survey software; developed by SOKKIA; was used to check data validity for blunders, missing observations, data continuity, and to create the network baselines. A preliminary data analysis was performed by using the same software to choose the optimum processing parameters. These parameters were used later for data processing. Then a single baseline differencing was carried out to get the most probable values (ΔX, ΔY, ΔZ) of each baseline separately, also the approximate coordinates of unknown points were obtained [5].

Network adjustment was then performed using three methods, namely:

A: Commercial software network adjustment.

B: GPS vector network adjustment.

C: Projected network adjustment.

In methods B, and C; Least squares theory (Variation of coordinates method) was chosen for adjustment processing starting with the approximate coordinates, adjustment matrices were formed by Microsoft Office Excel program, and the calculation operations were performed by using MATLAB program [2].

In the later method C, GPS baselines were projected on the UTM plane system [7], and then the network was adjusted as a traditional hybrid network. The error analysis was performed based on Variance-Covariance matrices. The pervious procedure can be outlined in the following flow chart, fig. 5.

3.3.1. Preliminary studies

Spectrum survey program; developed by SOKKIA; was used for analysis, processing and adjustment of the GPS data. The program allows user to control five variables, each of them was studied for each baseline separately to get the optimum values of each one. These variables are:

A: Interval time of observation.

B: Mask angle of elevation.

C: Maximum allowed DOP.

D: Observation duration.

E: Ephemeris corrections; (only SP3 format are applicable in the software).

Results show that the DOP and ephemeris corrections effect on short baselines is negligible. The most important results can be seen in figs. 6 – 8.

Table 1
Adjusted coordinates

Pt	Trilateration		Triangulation		Hybrid	
	X m	Y m	X m	Y m	X m	Y m
A	1000.000 ± 0.000	1000.000 ± 0.000	1000.000 ± 0.000	1000.000 ± 0.000	1000.000 ± 0.000	1000.000 ± 0.000
B	1000.000 ± 0.000	1301.538 ±0.002	1000.000 ± 0.000	1301.538 ± 0.000	1000.000 ± 0.000	1301.535 ± 0.002
C	1252.811± 0.002	1145.228 ± 0.003	1252.829 ± 0.003	1145.186 ± 0.004	1252.821 ± 0.001	1145.210 ± 0.002
D	1052.668± 0.008	628.538 ± 0.002	1052.622 ± 0.005	628.538 ± 0.008	1052.641 ± 0.003	628.539 ± 0.002

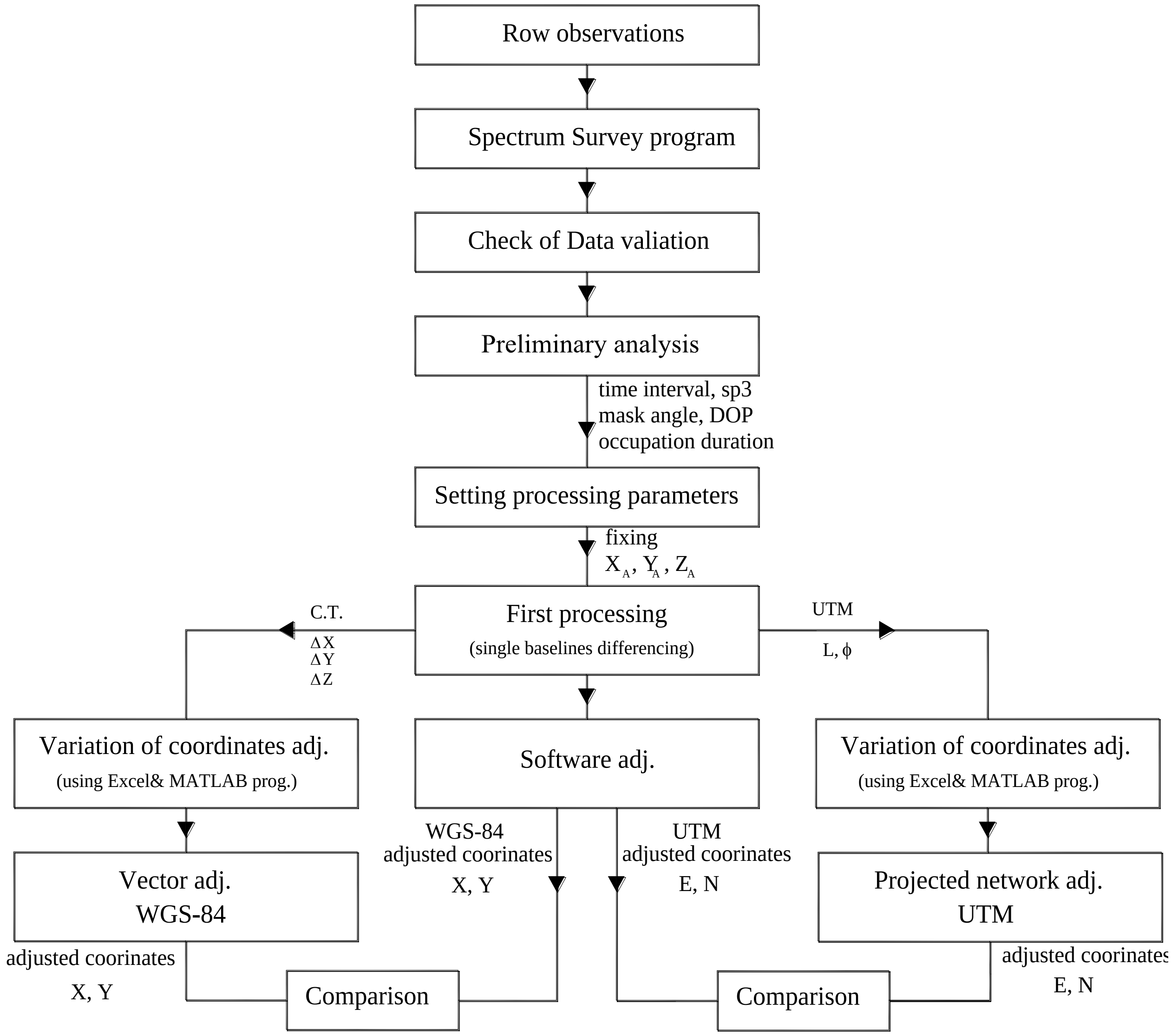
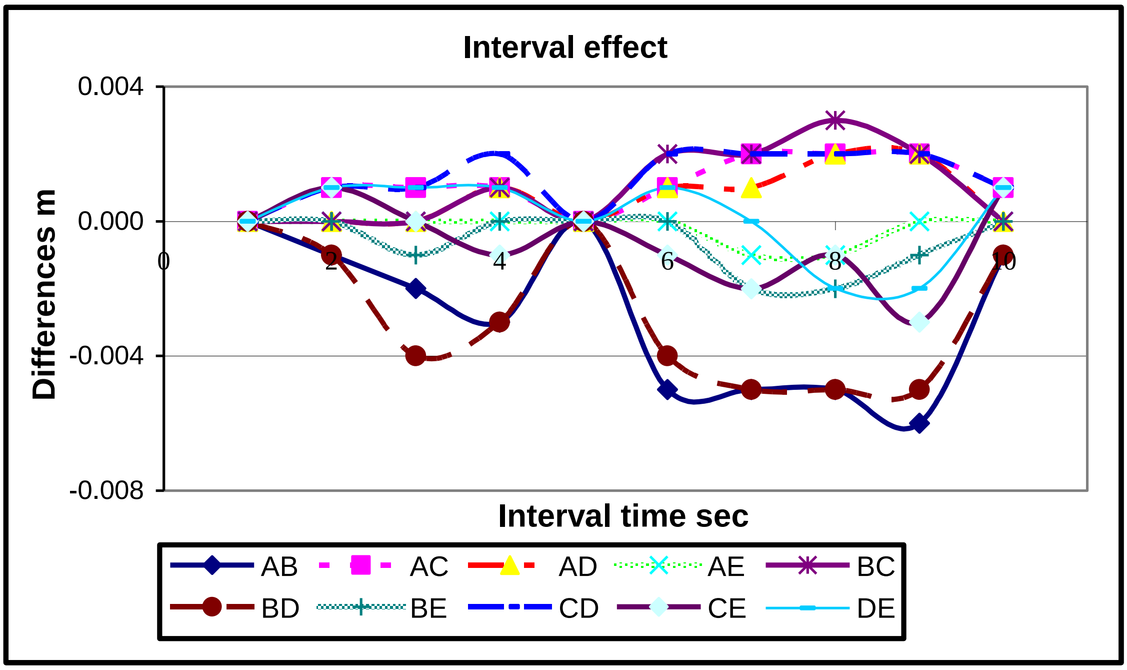
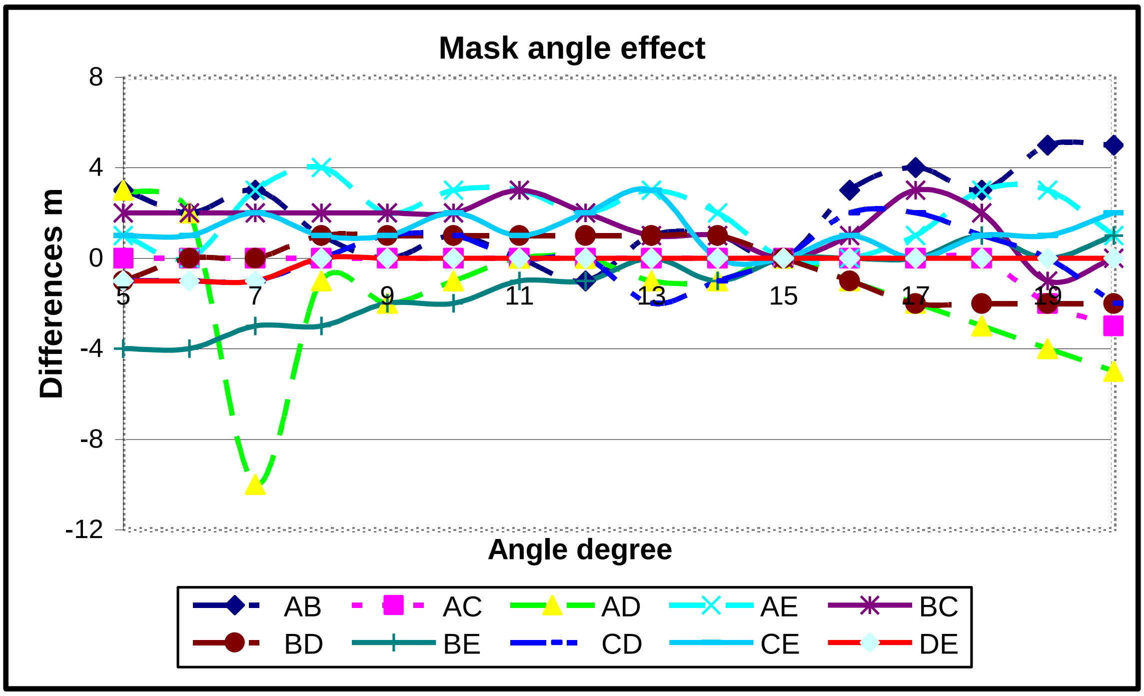


Fig. 5. GPS network processing and adjustment strategy.



* The Differences for each baseline is calculated relative to 5sec calculated length
* The pervious results at mask angle of 15° and DOP of 10

Fig. 6. Interval interpolation effect on GPS baselines.



* The pervious results at DOP = 10

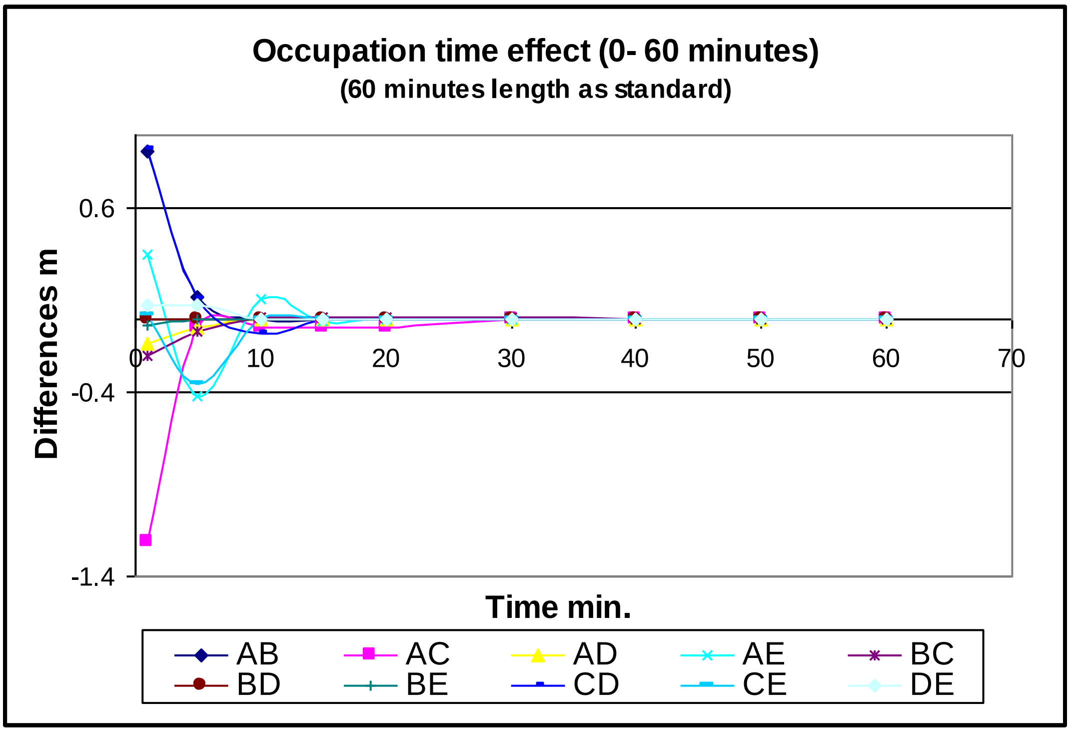
Fig. 7. Mask angle effect on GPS baselines.

3.3.2. Basic GPS Adjustment

The GPS network was adjusted as a free network by fixing point A as a reference station [5, 8]; the network was adjusted 3 times as follow:

A: Commercial software network adjustment.
B: GPS vector network adjustment.
C: Projected network adjustment.

The Spectrum survey program was used as strong commercial software, the results were expressed in WGS-84 (*X*, *Y*, *Z*) format, then they were transformed to UTM plane Coordinates (*E*, *N*) using the same software. A summary of the final software adjustment results are shown in tables 2, 3.



* The results are calculated for 60 minute duration, 15° mask angle, and DOP of 10

Fig. 8. Occupation Time effect on GPS baselines (one hour).

Table 2
Adjusted GPS coordinates, (WGS84)

	WGS84 coordinates		
Point	X m	Y m	Z m
A	4732331.575 ± 0.000	2723847.880 ± 0.000	3285478.283 ± 0.000
B	4732142.984 ± 0.005	2723979.180 ± 0.004	3285674.044 ± 0.006
C	4732215.306 ± 0.005	2724107.524 ± 0.004	3285414.741 ± 0.007
D	4732575.846 ± 0.005	2723736.933 ± 0.004	3285216.296 ± 0.006
E	4732496.937 ± 0.006	2723476.228 ± 0.005	3285529.639 ± 0.008

Table 3
Adjusted GPS coordinates (UTM)

	UTM coordinates	
Point	E m	N m
A	778595.833 ± 0.000	3456113.343 ± 0.000
B	778797.974 ± 0.006	3456337.074 ± 0.007
C	778880.755 ± 0.006	3456051.575 ± 0.007
D	778385.855 ± 0.006	3455802.419 ± 0.007
E	778189.417 ± 0.007	3456168.375 ± 0.009

4. Analysis and results

The networks adjustment results were analyzed. Firstly, as validation of results consistency, an internal accuracy analysis was performed by comparing the results of the different adjustment methods for both of traditional and GPS network separately. Secondly, a primary comparative analysis of traditional EDM baselines and single frequency GPS baselines was carried out. Thirdly, a basic comparative analysis for traditional and GPS networks was detailed. In this stage, a new network; namely Optimum network; was adopted as a reference standard for comparison. This new network was created by merging of the traditional and GPS adjustment results in one network based on the least squares similarity transformation principles.

4.1. Internal Accuracy Analysis

4.1.1. Traditional Network

The adjustment of the traditional network contains three types of procedures according to their observations: trilateration, triangulation and hybrid adjustment. The adjusted coordinates of hybrid adjustment were considered as standard for other traditional adjustment methods. Fig. 9, shows the positional standard deviation of the three methods. It can be seen that, the maximum standard deviation for all points is less than 10 mm for triangulation adjustment has the least precise results, while the minimum is less than 4 mm for hybrid adjustment.

The lengths differences were calculated in PPM format. Fig. 10, shows the differences between trilateration, triangulation and the hybrid baselines. It is noticed that the trilateration results is nearest to hybrid results with average difference of 5mm (16.3 PPM), while the average difference of triangulation results was 7mm (19.3 PPM).

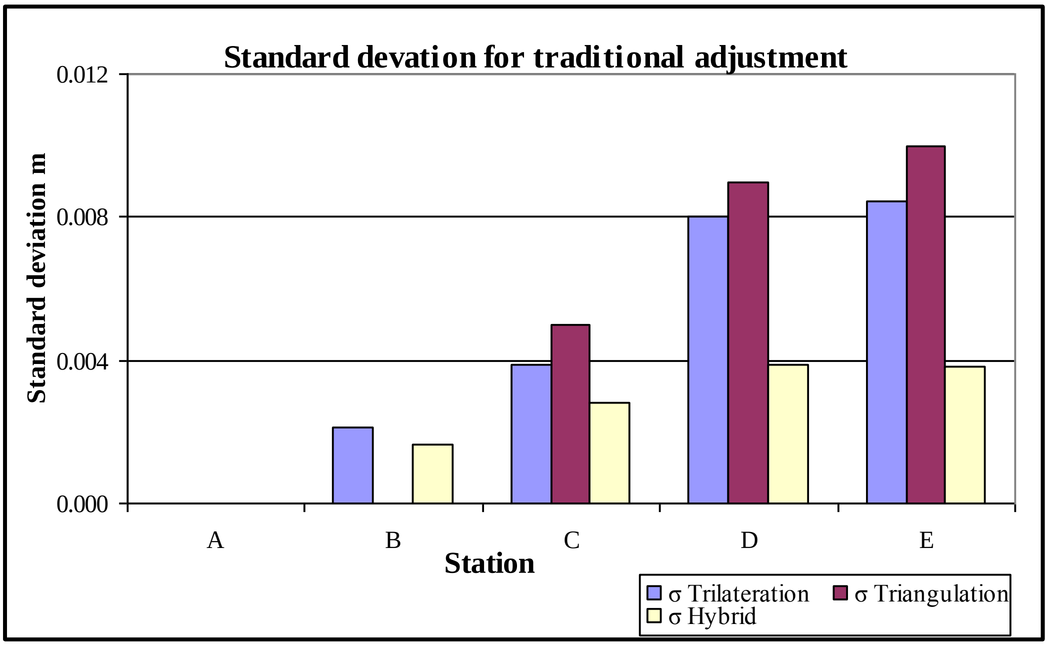
4.1.2. GPS network

The adjustment of the GPS network was carried out using three methods namely: Commercial software adjustment, GPS vectors adjustment and Projected network adjustment. The used commercial software; Spectrum Survey software; is designed for high-order geodetic surveys, and outputs various statistics to assist in blunders detecting. So, the software adjustment results will be considered as standard for the other adjustment methods. The comparison was performed in two different coordinate systems as follows:

- Software adjustment and GPS vector adjustment results were compared in WGS84 system.
- Software adjustment results and Projected network adjustment results were compared in the UTM plane system.

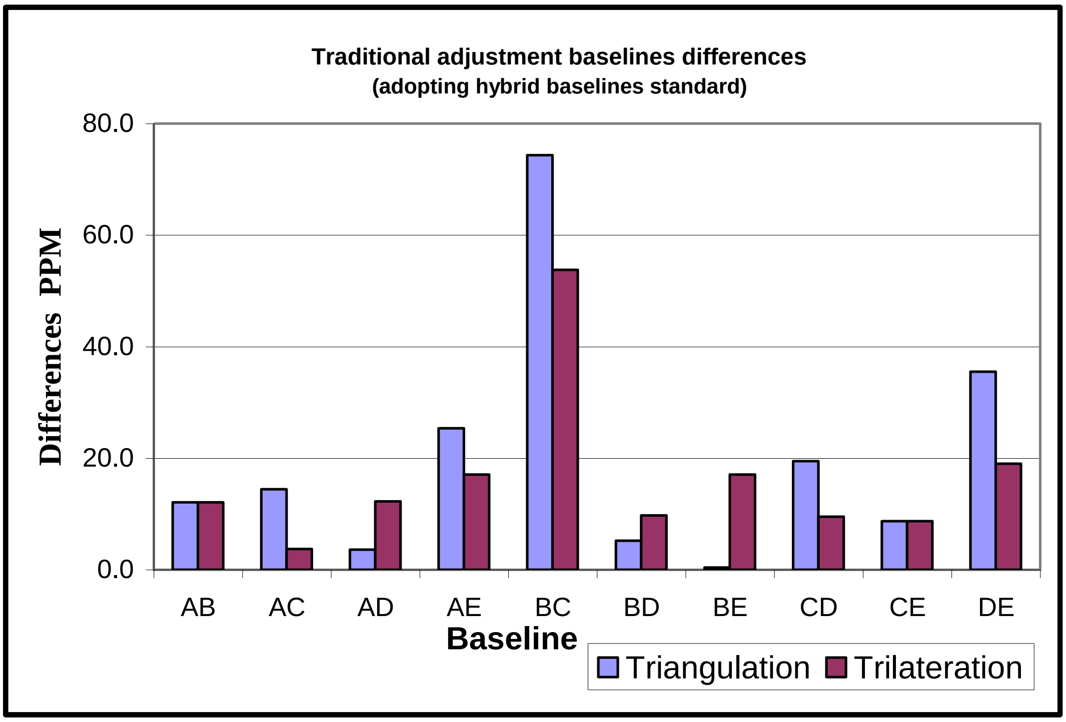
Fig. 11 shows the two dimensional and three dimensional positional differences between the software adjustment and the GPS vectors adjustment methods adopting the software results as standard. It shows that the two methods results are consistent with maximum horizontal position difference less than 6 mm, the vertical differences are almost negligible also.

This difference represents an average baseline relative error about 13 PPM. This small error indicates that the GPS vector adjustment results are consistent with the GPS commercial software results. It can be concluded that the GPS geodetic networks can be adjusted using the personal software based on variation of coordinates principles without considerable differences from the expensive sophisticated GPS adjustment software.



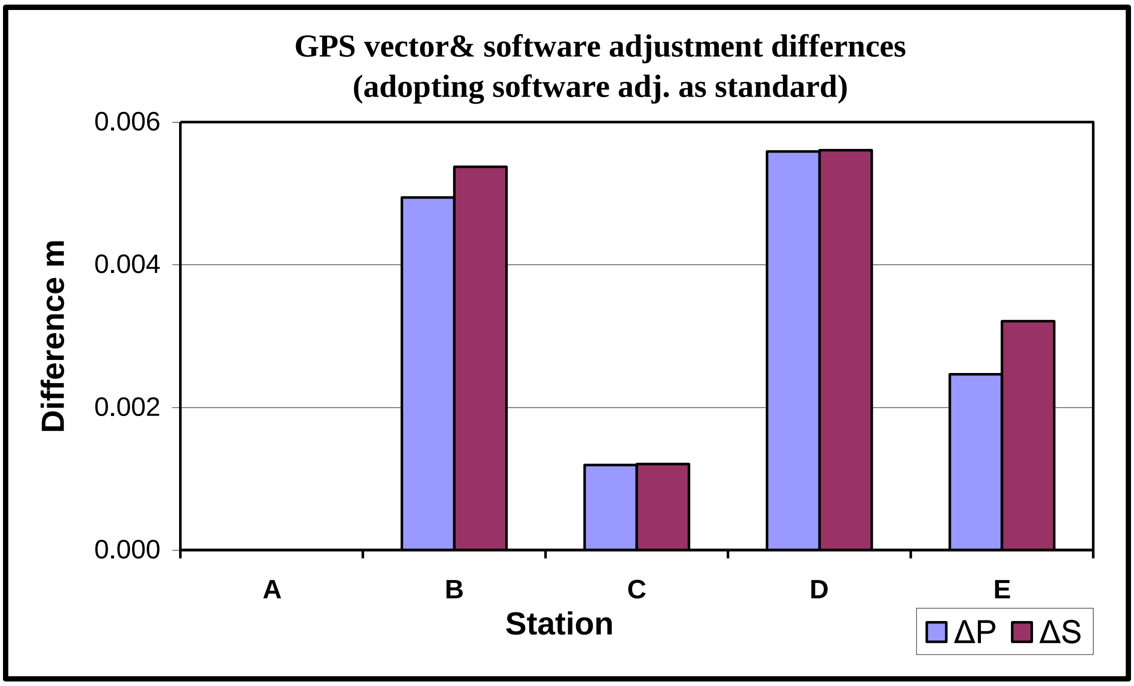
* σ is the positional standard deviation $= \sqrt{\left((\sigma x)^2 + (\sigma y)^2\right)}$

Fig. 9. Accuracy of traditional adjustment coordinates.



* $PPM = \dfrac{Difference}{Hybrid\ length}$

Fig. 10. Comparison of traditional adjusted baselines.



* ΔP is the horizontal positional difference $= \sqrt{\left((\Delta x)^2 + (\Delta y)^2\right)}$

* ΔS is the vector positional difference $= \sqrt{\left((\Delta P)^2 + (\Delta z)^2\right)}$

* Δx, Δy is the positional difference relative to the software adjustment

Fig. 11. Comparison of software and GPS vectors adjustment coordinates.

4.2. Basic comparative analysis

4.2.1. Geometrical consideration

Control networks are concerned with accurate determination of point positions. The geometrical elements of such determination are usually expressed in the form of four distinct elements; shape, scale, orientation, and position, [9].

It should be noted that although GPS is basically a positioning system, it can not be used alone for determination of precise position on the required geometric accuracy standards, the Differential GPS (DGPS) can be adopted for that. The DGPS produces GPS vectors for pairs of stations; lines; in the networks. In this case only the first three geometrical elements can be accurately determined: shape, scale, and orientation. For accurate determination of positions, some points (at last one point) should be available with known coordinates.

4.2.2. Comparison criteria

To conduct the basic comparison; previously mentioned, five networks will be considered:

Network A: the optimum network; (as standard for all comparisons).
Network B: the adjusted GPS network.
Network C: transformed trilateration network; (applying rotation and translation).
Network D: transformed triangulation network; (as network C).
Network E: transformed hybrid network; (as network C).

4.2.3. The optimum network (network A)

The optimum network was generated by applying the two dimensional transformation between the traditional hybrid network, and the GPS network using the GSP as distinguished target. The optimum transformation parameters were determined using the least square theory. The final transformed coordinates of the optimum network were computed as tabulated in table 4.

Table 4
Optimum network coordinates

Point	UTM Coordinates	
	E	N
A	778595.833	3456113.343
B	778797.986	3456337.082
C	778880.777	3456051.594
D	778385.860	3455802.427
E	778189.439	3456168.386

4.2.4. The comparison networks (networks B, C, D, and E)

The GPS network is already oriented with the UTM system. however, The traditional trilateration, triangulation, and hybrid networks was reoriented to the UTM reference direction by rotating the network with angle θ, which will make one baseline of the traditional network coincide with the same baseline in the GPS network. The rotation will be performed by applying rotation transformation.

Since there were 10 baselines in the network, there were 10 solutions for rotation problem, an additional solution was computed by rotating the network with angle θ_{av}; the average rotation angles. The later case was considered for the basic comparisons.

A: Scale comparison

The scale comparison is usually used for trilateration, hybrid, and GPS networks. The comparison will involve the lengths of networks (B, C, and network D), fig. 12. It can be noticed that, the GPS network has the nearest results to the optimum network, then the Trilateration network.

B: Shape comparison

The shape comparison is usually used for trilateration, hybrid, and GPS networks. The comparison will involve the angles of networks (A, B, and network C), fig. 13. It can be noticed that, the GPS network has the nearest results to the optimum, then the triangulation network.

C: Orientation comparison

The orientation comparison is used for trilateration, hybrid, and GPS networks. Orientation discrepancies were obtained and was found that, the GPS network average angular difference was 10 PPM, with standard error of 6.9 sec which shows that the differences between GPS and optimum network is also inconsiderable.

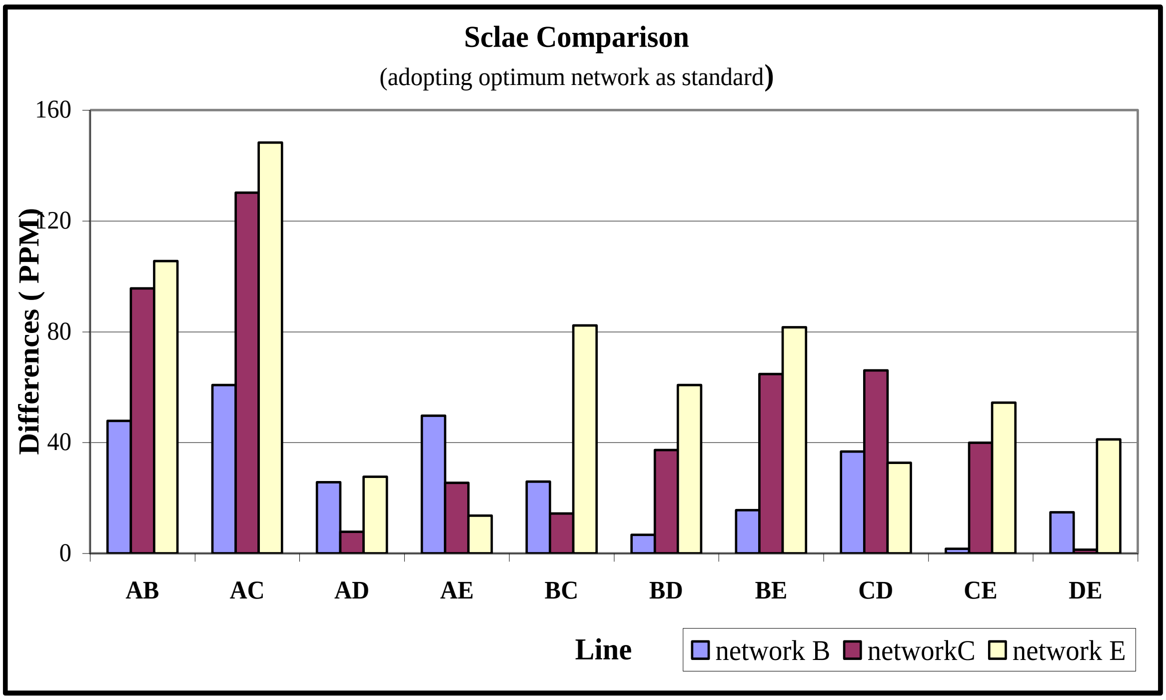


Fig. 12. Scale comparison of networks B, C, and E (adopting A as standard).

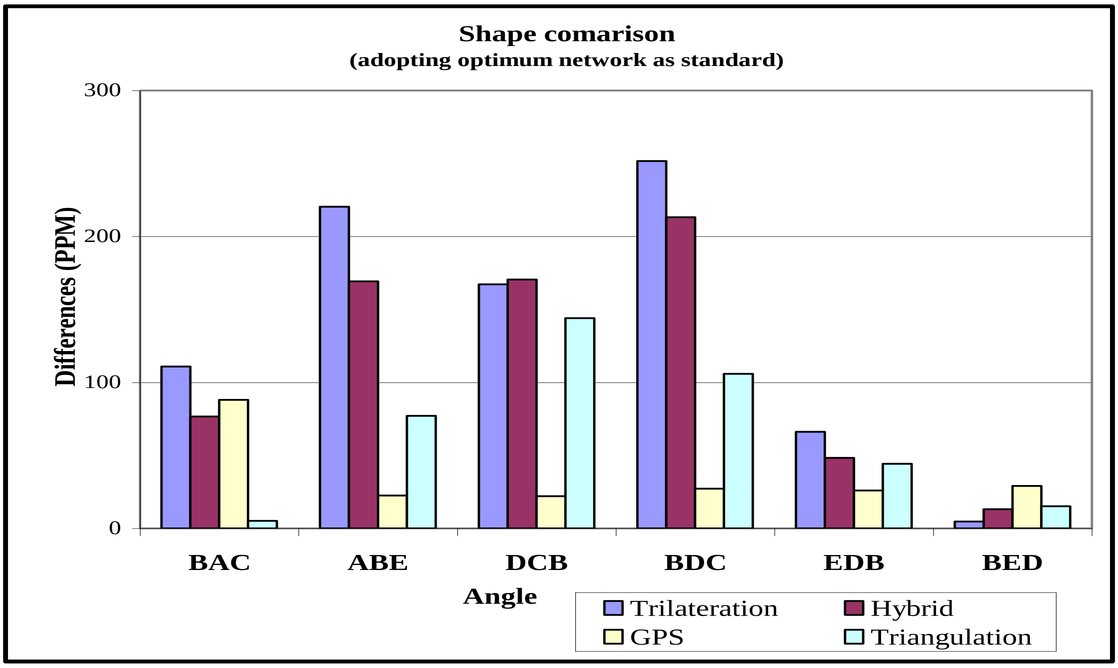


Fig. 13. Shape comparison of networks B, C, and E (adopting A as standard).

5. Conclusions and recommendations

Based on the investigations reported herein and the obtained results from this research, the following major findings and conclusions can be laid down:

1- Single frequency DGPS can be used for ordinary geodetic or engineering surveying instead of traditional EDM measurements with average relative differences about 30 PPM.

2- It is recommended to use static GPS observation for more than 40 minutes, mask angle from 9^o to 16^o with DOP values between 4 and 6 for the minimum interval time, and there is no need to use special ephemeris corrections. The surveyor may choose the appropriate value according to the site conditions, and satellites visibility.

3- The comparison between using personal software; developed in the present work; in GPS vectors adjustment and using commercial software, clearly demonstrate that the two methods have relative differences less than 13 PPM. That indicates that the GPS geodetic networks can be adjusted using personal software based on variation of coordinates principles without considerable differences from expensive sophisticated GPS adjustment software.

4- For the suggested projected network adjustment technique, the comparison between using personal software and commercial software, clearly demonstrate that the two methods have relative differences less than 10 PPM. That indicates that the suggested technique can be used for the adjustment of GPS networks without considerable differences.

5- GPS observations can be used for reorienting the traditional network measurements to a global system by applying rotation and translation by which, one or more common baseline in the local network will be coinciding with its equivalent GPS baseline/ baselines. The selection of the fixed baselines will results in very small discrepancies. It is recommended to adopt the maximum available number of common baselines.

6- Combining the adjusted GPS network results with the hybrid network results using least squares transformation principle resulted in new network, named (The Optimum Network) which can be considered to be more strong because it takes he advantages of the both networks. Therefore, in case of existing old control points in the site, it is recommended to combine their coordinates with the new GPS coordinates in a new optimum network using least squares transformation principle as illustrated in the research.

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