GPS Based Traversing for Topographic-cadastral Surveying and Staking

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Key words: GPS, traversing, staking, cadastral surveying, topographic surveying, permanent GPS stations

SUMMARY

Practical engineering applications for determination of observation stations for topographic and cadastral surveys and staking application are required traversing. GPS one of the efficient and optimum surveying techniques has been using routinely for positioning of the traverse points among the satellite-based surveying. In our days, many surveyor use GPS directly or undirectly for supporting other terrestrial and conventional methods. The aim of this study is the solution of traversing problem by different GPS survey strategy. These strategies have been applied in a test area by utilizing a traverse points. First of all, each points use for study positioned from IGS and EUREF permanent stations by using long session interval with high precision for comparison. After, the study focuses on and probes, GPS observation of traverse points from the nearest referance points by kinematic methods. As the second alternative strategy, we used permanent stations in Istanbul for kinematic positioning. Detailed comparisons and suggestions are given for each strategy as accuracy and economy. Several conclusions are emphasized.

SUMMARY IN TURKISH

Pratik mühendislik uygulamalarında, topografik, kadastral ölçmeler ve aplikasyon çalışmalarında sabit ölçü noktası olarak kullanılan noktalar genellikle poligonasyon ile konumlandırılmaktadır. GPS, uydulara dayalı konumlama teknikleri içerisinde oldukça etkili, verimli ve rutin olarak kullanılmaktadır. Günümüzde, bir çok ölçmeci GPS'i doğrudan veya diğer yersel ve klasik ölçmeleri desteklemek amacıyla dolaylı olarak kullanmaktadır. Bu çalışma poligonasyon probleminin farklı bir GPS ölçme stratejisi ile çözülmesini amaçlamaktadır. Bir test alanına poligon noktaları tesis edilerek bu stratejiler araştırılacaktır. İlk olarak her poligon noktası, karşılaştırma amacıyla, GPS ile uzun gözlem aralıklarında yapılan statik oturumlar ile IGS ve EUREF sabit GPS istasyonları kullanılarak hassas bir şekilde konumlanmıştır. Daha sonra poligon noktaları çalışma bölgesine en yakın bir referans noktası kullanılarak kinematik gözlemlerle konumlandırılmıştır. İkinci alternatif olarak İstanbul bölgesi içerisinde ve çevresinde bulunan çalışma bölgesine faklı uzaklıklarda olan sabit GPS noktaları kullanılarak kinematik konumlama gerçekleşmiştir. Söz konusu yaklaşımlardan elde edilen sonuçlar irdelenmiştir. Detaylı karşılaştırma ve öneriler sunularak her strateji, doğruluk ve ekonomi kriterleri de göz önünde tutularak irdelenmiş ve çeşitli öneriler vurgulanmıştır.

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1. INTRODUCTION

It is known that geodetic GPS surveying based on relative positioning principle. Simultaneously observation of pseudorange and carrier phase measurement from same satellite gruop is processed for this purpose (Hoffman-Wellenhof et all, 1997; Leick, 1990). This application required minimum two or more GPS receiver. When considering the price of the receivers, it is restricted that the use of GPS technique efficiently. However, maybe the most rationalist solution is to become widespread that permanent GPS station established by several institutions. To start from this approach, in this paper, it is examined that use of permanent station for traversing in practice engineering surveying (Aydın, Soycan, 2004; Aydın, Soycan M., Soycan A., 2004; Hu, Khoo, Goh, Law, 2005; Soycan M., Soycan A., 2002; Soycan M., Soycan Topbaş A., 2002).

Seven pillar sites as new points, IGS and EUREF permanent GPS stations (http://igs.ifag.de, http://igscb.jpl.nasa.gov), Tübitak Marmara Research Center permanent stations AVCT (http://www.nemrut.mam.gov.tr/research/gps/project/project.html) used in test area for the study. Positions of the new points determined by using strategies that shown in Figure 1.

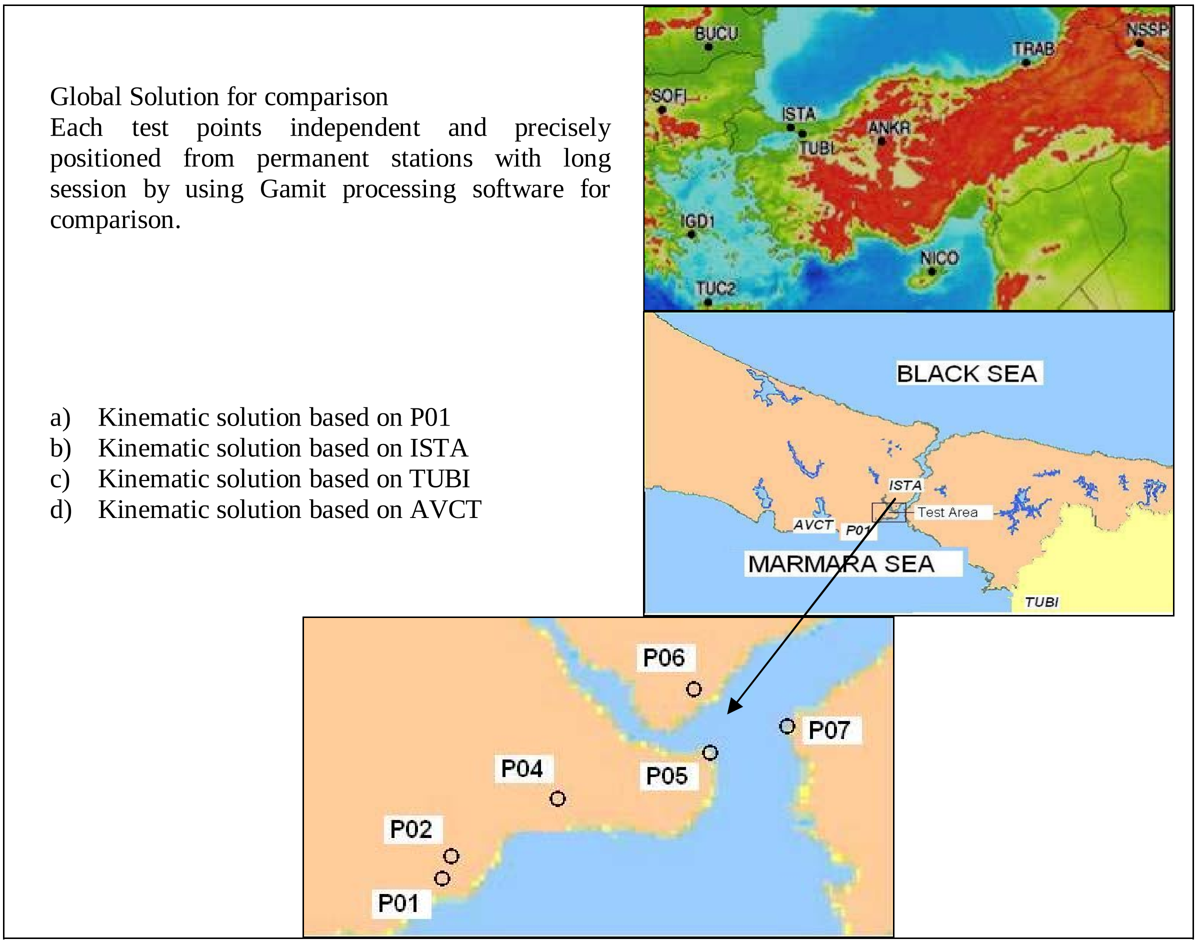


Figure.1. Test Area and GPS Strategy for Traverse Point Positioning

Firstly, inner accuracy of each strategy will be examined. The coordinates values obtained from first strategy will taken fixed. It means that the coordinates value obtained from first strategy assumed as exact value and coordinates value obtained from other methods will be compared to the first solution for determination of their accuracy and consistency.
In research; Trimble 4000SSI GSI, Trimble 4000SSE geodetic surveyor, Ashtech Z Surveyor GPS Receivers, Ashtech Solution 2.60 and Gamit GPS processing software were used.

2. GLOBAL NETWORK SOLUTION FROM IGS AND EUREF STATIONS FOR COMPARISON

The GPS observation was realized by recording L1-L2 frequency and by choosing 15 seconds record interval and 15^{o}-satellite elevation mask with geodetic GPS hardware given above in static mode. On different 5 test points, simultaneously sessions changing between 3 and 5 hours were done. The observations, ephemeris and meteorologic data files that collected, so GPS measurements were convert to RINEX format before the post-processing. The GPS datas which archived at observation dates of permanent GPS stations (IGS and EUREF) and the final precise IGS ephemeris files related to GPS weeks and the ITRF 2004.944 coordinates that is the observation epoch of stations, were recorded after downloading from SOPAC, BKG and IGS analysis centers by using web.

All of the test points were seperately tied to permanent GPS stations by using GAMIT softwere. Thus, the baseline vector between permanent GPS stations-test point were processed. For example, 6 differant baseline vector between permanent GPS stations (ISTA,TUBI,TRAB,NICO,SOFI,BUCU), were used for positioning the point P02. Then the accuracy of the baseline vectors were researched by depending on statistical information obtained from processing results. With this research the baseline vectors between the points, which's simultaneously static GPS observations have done and ΔX, ΔY, ΔZ baseline components with their standard deviation, were calculated with 95 % statically confidence.

Tablo:5 Examination of Constrained Adjustment Results

Number of fixed point	6			
Number of total points	16			
Number of vectors	60			
Number of vector components	180			
Degree of freedom	150			
	Max.	Min.	Mean	RMS
Standard deviation of baseline vector componenet	0,0460	0,0039	0,0161	0,0192
Residuals resulting from free adjustment	0,0139	-0,0141	-0,0001	0,0046
Standart deviation of X coordinates resulting from free adjustment	0,0041	0,0016	0,0023	0,0024
Standart deviation of Y coordinates resuting from free adjustment	0,0038	0,0010	0,0016	0,0018
Standart deviation of Z coordinates resuting from free adjustment	0,0065	0,0016	0,0029	0,0031
Residuals resulting from constrained adjustment	0,0253	-0,0223	-0,0010	0,0080
Standart deviation of X coordinates resuting from constrained adjustment	0,0040	0,0026	0,0033	0,0034
Standart deviation of Y coordinates resuting from constrained adjustment	0,0027	0,0019	0,0023	0,0023
Standart deviation of Z coordinates resuting from constrained adjustment	0,0045	0,0031	0,0039	0,0039
Scale differences between free and constrained adjustment	0,02ppm			

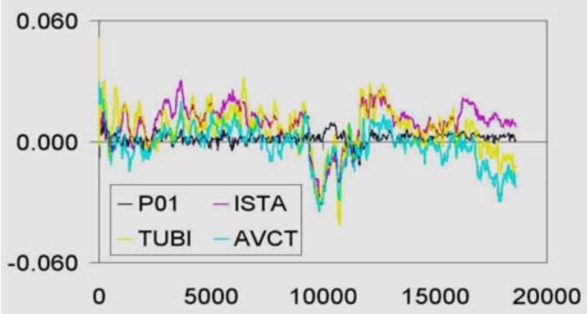
For computing the coordinates of the test points of research, first of all, independent baseline vectors were adjusted freely so blunder and outlier searching was made. Than freely adjusted baselines have been readjusted as constrained by fixing that the ITRF 2004.944 coordinates which are the observation epoch of the permanent stations ISTA, TUBI, TRAB, NICO, SOFI, BUCU, and all test points was positioned on this epoch. Several statistical informations were given at Table 5 as the result of this study.

3. KINEMATIC SOLUTION BASED ON FOUR DIFFERENT REFERENCES

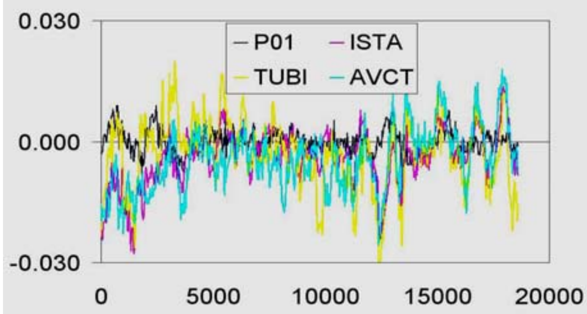
In the following stage of research, each test points have been positioned with 3-5 hours GPS observations based on ISTA, AVCT, TUBI and P001 references. The achivening GPS data have been processed epoch by epoch as kinematic mode. The point positions computed by this approach have been compared with the precise coordinates values obtained from global network solution. By examining the differences for horizontal and vertical components an accuracy of each strategy have been determined. In this investigation the effects of distance to reference for each test points.

P02

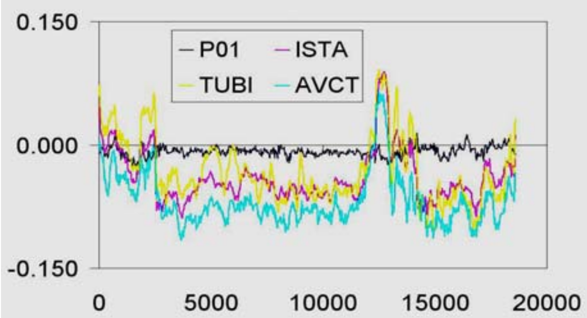
Latitude Differences (m)



Longtitude Differences (m)



Elipsoidal Height Differences (m)



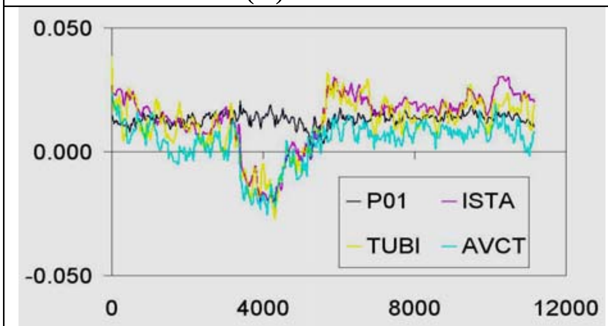
2 and 3 Dimensional Positional errors(units are metre)

References		P01	ISTA	TUBI	AVCT
Distances to references		498	14746	50447	16443
3D	max.	0.0246	0.1044	0.1191	0.0928
	min.	0.0004	0.0035	0.0042	0.0113
	avr.	0.0094	0.0417	0.0778	0.0420
2D	max.	0.0117	0.0349	0.0343	0.0417
	min.	0.0001	0.0006	0.0011	0.0004
	avr.	0.0035	0.0209	0.0342	0.0234

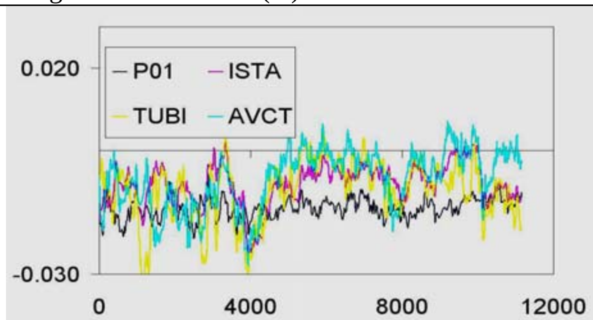
Figure.2. Latitude, Longtitude and Ellipsoidal Height Differences between Global Network Solution and Kinematic Solution based on four different reference stations for P02

P04

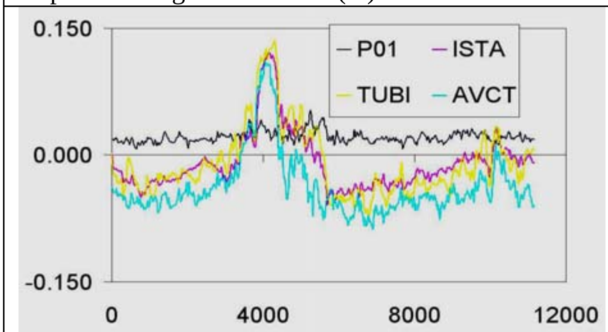
Latitude Differences (m)



Longtitude Differences (m)



Elipsoidal Height Differences(m)



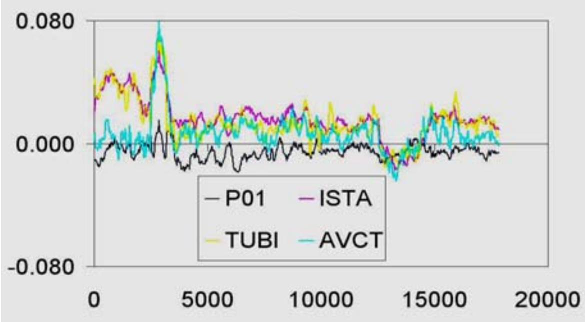
2 and 3 Dimensional Positional errors(units are metre)

References		P01	ISTA	TUBI	AVCT
Distances to references		3048	12429	48996	18870
3D	max.	0.0253	0.1245	0.1160	0.1384
	min.	0.0010	0.0065	0.0088	0.0068
	avr.	0.0160	0.0353	0.0657	0.0453
2D	max.	0.0163	0.0394	0.0335	0.0362
	min.	0.0006	0.0031	0.0040	0.0127
	avr.	0.0103	0.0186	0.0295	0.0194

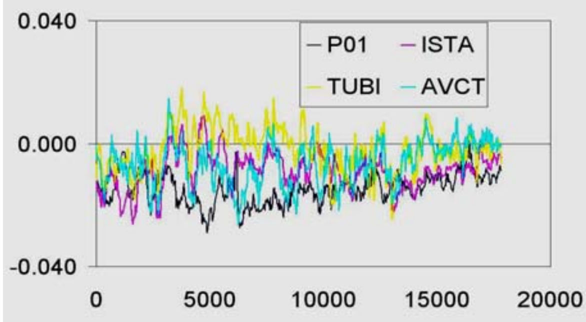
Figure.3. Latitude, Longtitude and Ellipsoidal Height Differences between Global Network Solution and Kinematic Solution based on four different reference stations for P04

P05

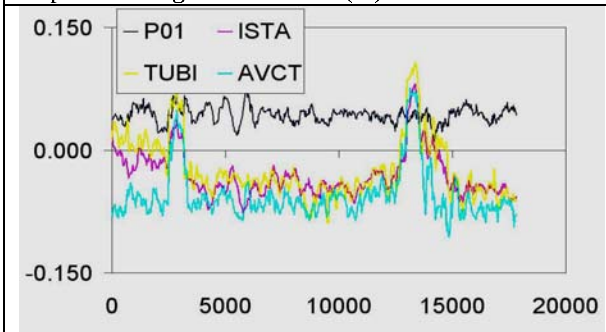
Latitude Differences (m)



Longtitude Differences (m)



Elipsoidal Height Differences (m)



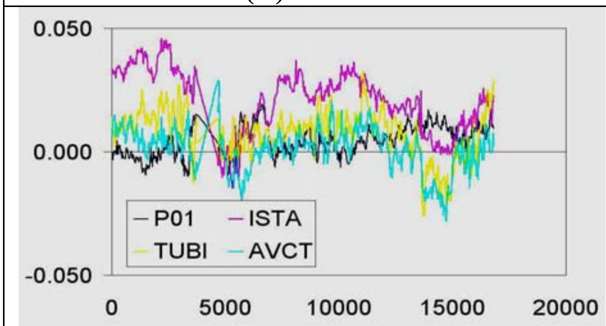
2 and 3 Dimensional Positional errors(units are metre)

References		P01	ISTA	TUBI	AVCT
Distances to references		6488	10178	46658	22334
3D	max.	0.0343	0.0721	0.1086	0.1078
	min.	0.009	0.0179	0.0077	0.0029
	avr.	0.0264	0.0367	0.0634	0.0470
2D	max.	0.0327	0.0632	0.0803	0.0709
	min.	0.0011	0.0070	0.0002	0.0005
	avr.	0.0138	0.0217	0.0280	0.0216

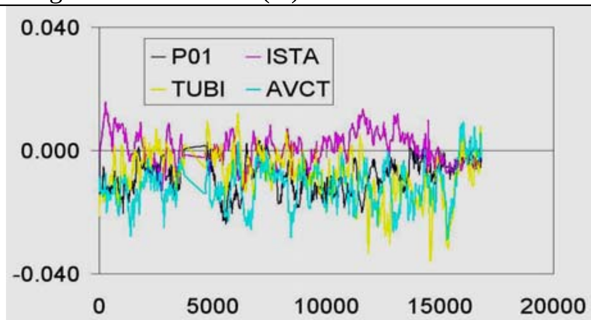
Figure.4. Latitude, Longtitude and Ellipsoidal Height Differences between Global Network Solution and Kinematic Solution based on four different reference stations for P05

P06

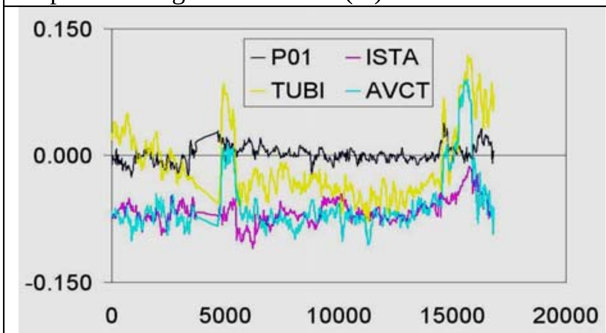
Latitude Differences (m)



Longtitude Differences (m)



Elipsoidal Height Differences (m)



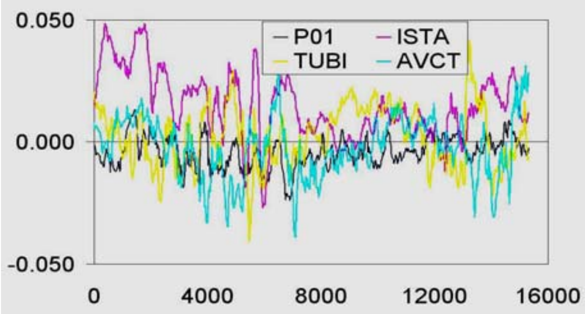
2 and 3 Dimensional Positional errors(units are metre)

References		P01	ISTA	TUBI	AVCT
Distances to references		6840	9033	47713	22172
3D	max.	0.0401	0.1271	0.1110	0.1072
	min.	0.0043	0.0059	0.0143	0.0048
	avr.	0.0280	0.0349	0.0704	0.0480
2D	max.	0.0246	0.0456	0.0964	0.0445
	min.	0.0014	0.0001	0.0018	0.0012
	avr.	0.0160	0.0215	0.0292	0.0215

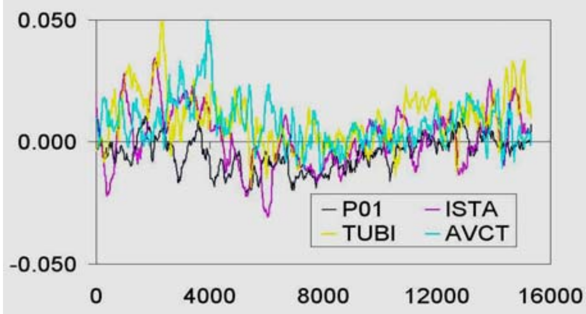
Figure.5. Latitude, Longtitude and Ellipsoidal Height Differences between Global Network Solution and Kinematic Solution based on four different reference stations for P06

P07

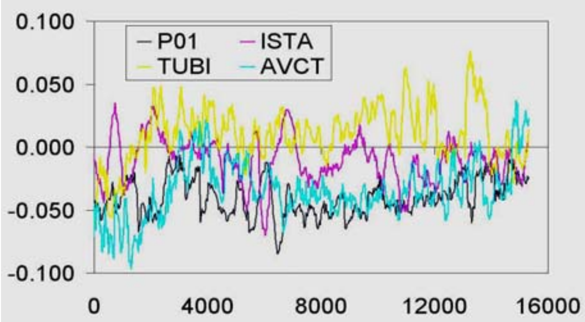
Latitude Differences (m)



Longtitude Differences (m)



Elipsoidal Height Differences (m)



2 and 3 Dimensional Positional errors(units are metre)

References		P01	ISTA	TUBI	AVCT
Distances to references		8247	9293	45587	24071
3D	max.	0.0428	0.0848	0.0976	0.0799
	min.	0.0011	0.0024	0.0080	0.0020
	avr.	0.0310	0.0343	0.0615	0.0483
2D	max.	0.0309	0.0571	0.0533	0.0547
	min.	0.0003	0.0005	0.0003	0.0008
	avr.	0.0198	0.0214	0.0250	0.0219

Figure.6. Latitude, Longtitude and Ellipsoidal Height Differences between Global Network Solution and Kinematic Solution based on four different reference stations for P07

The latitudes, longtitudes and ellipsoidal height differences between positions precisely obtained based on IGS and EUREF permanent GPS stations and positions obtained from epoch by epoch kinematic solution P01, ISTA, TUBI and AVCT for each test points have been presented Figure 2,3,4,5,6. By using these differences 2 and 3 dimensional positional errors have been computed. Maksimum, minimum and avarage value of 2 and 3 dimensional positional errors were determined for each test points.

It has seen that high corelation between positional errors and the distance between test points and reference points. A graphic was prepared for 2 and 3 dimensional positional errors and the distance between test points and reference points.

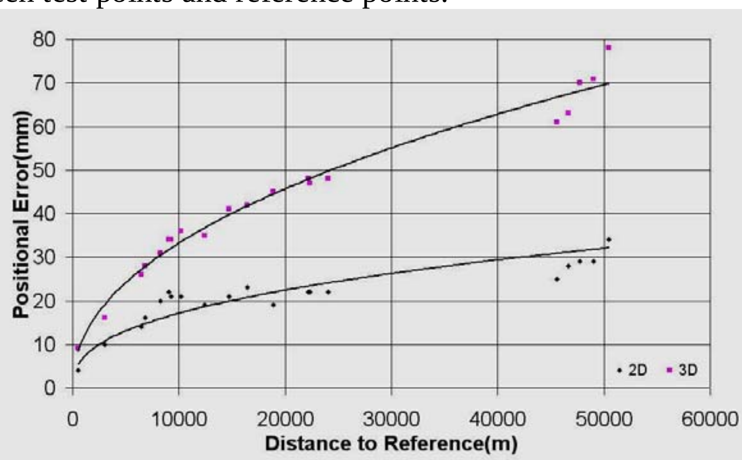


Figure.7. 2 and 3 dimensional positional errors as depending on distance to reference.

According to data on graphic; it is possible to obtained 7mm 2 dimensional, 15mm 3 dimensional positional error by using a referance with 1km lenght. When using a refence with 10km lenght, 18mm and 33mm positional errors can be availeble respectively for 2 and 3 dimension. For a refence with 50'km lenght, these errors increase 32mm and 70mm.
The difference between 2 dimensional positional errors and 3 dimensional positional errors increase as depending on distance between reference and test points. The most important reason of this problem is negative effects on the height components due to atmosphere. This case can be seen clearly in Figure 2,3,4,5,6.

4. CONCLUSION

As a result of all examination and evaluation, it can be said that, traversing by kinematic GPS methods from one reference has enough accuracy for topographic, cadastral surveying and staking application in practice (Aydın, Soycan, 2004; Aydın, Soycan M., Soycan A., 2004; Hu, Khoo, Goh, Law, 2005; Soycan M., Soycan A., 2002; Soycan M., Soycan Topbaş A., 2002). This method is very simple, economic and less time consuming as to other surveying methods, moreover it is possible to use a reference with 50 km lenght. For this purposes, ıt was not necessary to do an expensive work like building vertical and horizontal network, if the stations that provides the expecting accuracy for geodetic aims could be built in urban areas and sufficient densification. However the point, which's coordinates were known accurately and quality as it is expected were found fastly. Consequently an active usage of the tools that is united of the totalstation and GPS receiver will be provided for geodetic measurement for details and topographic aim.

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