

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**COMPARISON OF SATELLITE POSITIONING TECHNIQUES ON
UNMANNED AERIAL VEHICLE BASED PHOTOGRAMMETRY**



M.Sc. THESIS

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Department of Geomatics Engineering

Geomatics Engineering Programme

DECEMBER 2019

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**İNSANSIZ HAVA ARACI İLE FOTOGRAMETRİDE UYDU
KONUMLAMA TEKNİKLERİNİN KARŞILAŞTIRILMASI**

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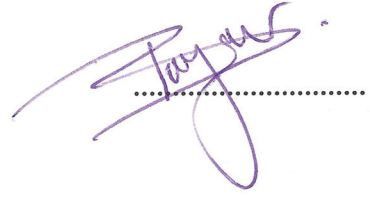
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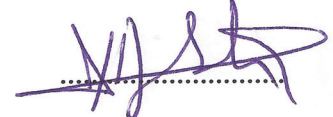
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To my family and friends



FOREWORD

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ABBREVIATIONS

C/A	: Coarse/Acquisition
DG	: Direct Georeferencing
ECEF	: Earth-Centered Earth-Fixed
GCP	: Ground Control Point
GDOP	: Geometric Dilution of Precision
GNSS	: Global Navigation Satellite System
GPS	: Global Positioning System
GRS 80	: Geodetic Reference System 80
GSD	: Ground Sample Distance
IERS	: International Earth Rotation and Reference Systems Service
IGS	: International GNSS Service
IMU	: Inertial Measurement Unit
IRM	: IERS Reference Meridian
IRP	: IERS Reference Pole
ISKI UKBS	: Istanbul Su Kanalizasyon Idaresi-Uydu Konum Belirleme Sistemi
ITU	: Istanbul Technical University
ITRF	: The International Terrestrial Reference Frame
İHA	: İnsansız Hava Aracı
LSMSDPR	: Large Scale Maps and Spatial Data Production Regulation of Turkey
N-RTK	: Network-Real Time Kinematic
PDOP	: Position (3D) Dilution of Precision
PPK	: Post Process Kinematic
PPP	: Precise Point Positioning
RTK	: Real Time Kinematic.
SfM	: Structure from Motion
SPP	: Standard Point Positioning
UAV	: Unmanned Aerial Vehicle
UVS	: Unmanned Vehicle Systems
WGS 84	: World Geodetic System 84
VDOP	: Vertical Dilution of Precision

VLBI : Very Long Baseline Interferometry



SYMBOLS

λ, Θ, r	: Polar Coordinates
X, Y, Z	: Axis
x, y, z	: Ellipsoidal Cartesian Coordinates
φ, λ, h	: Ellipsoidal Curvilinear Coordinates
N	: Radius of curvature
n, e, u	: Local coordinates
X_i	: Position Vector
a	: Semi-major axis of ellipsoid
b	: Semi-minor axis of ellipsoid
$\varepsilon, \psi, \omega$	: Axis rotations in right-hand coordinate system
$1+\Delta$	: Scale factor
P	: Pseudorange
Φ	: Carrier phase
O	: Principal point
f, c	: Focal length
M	: Rotation matrix
v	: Residuals



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COMPARISON OF SATELLITE POSITIONING TECHNIQUES ON UNMANNED AERIAL VEHICLE BASED PHOTOGRAMMETRY

SUMMARY

By the end of the 20th century, UAVs (Unmanned Aerial Vehicle) were started to widely used on daily life in consequence of decreased production cost and various options. UAV essentially is remotely controlled or autonomous aircraft that performs particular tasks and can carry payload without pilot onboard. While comparing with full size manned air vehicle, UAVs reduce costs and give more flexibility in terms of take-off area and size. Therefore, it is highly demanded in the market by the agriculture, forest, mine, construction, archeology, architectural areas. On the other hand, capability of carrying camera onboard enables to make use of it in photogrammetric technique. Conventional photogrammetric applications, which commonly used for mapping and modelling large and remote areas, is very costly in terms of carrier, labor and ground services. Hence, UAV is increasingly preferred instead of full size aerial vehicles in many applications depending on the purpose of applications.

Main problem of photogrammetry is defining unknown parameters of collinearity equations these constitute mathematical model. These unknowns are interior and exterior orientation parameters. Photogrammetric cameras interior parameters are generally known. In conventional airborne photogrammetry, exterior orientation parameters are computed by ground control points (GCP), which are signaled and measured on the ground. Setting GCPs on the vast areas are time consuming and costly. So, it is preferred air triangulation method to reduce number of GCPs with bundle block adjustment.

By development of sensor systems, UAVs are oriented by their own navigation system consists of GNSS (Global Navigation Satellite System) and IMU (Inertial Measurement Unit). Integration of GNSS-IMU with UAV brings up new approaches in photogrammetry. Obtaining instantaneous coordinates and rotation parameters can be utilized for orientation of images. Importing exterior orientation parameters from sensors makes process much less costly and labor intensive. It is enhancing accuracy of points to apply in Air triangulation. The process including orienting image and getting global geodetic coordinates of images by GNSS data, is called "Direct Georeferencing". Moreover, hand, initial exterior orientation values provided by GNSS and IMU improves accuracy of bundle block adjustment. Depending on GNSS measurement technique, accuracy of the system can be increased employing less ground points.

Although classical absolute GNSS measurement gives low accuracy, Real Time Kinematic (RTK), Post Process Kinematic (PPK) as relative and Precise Point Positioning (PPP) as absolute positioning approaches enhance positioning accuracy up to cm level. In RTK measurements, rover takes corrections via radio connection from at least one station that's position is known precisely and makes continuously observation. During the GNSS survey, receiving corrections data simultaneously

reduces office works. That is why it is projected that integration of RTK with UAV photogrammetry can improve credibility of the measurements.

In this study, it is intended to analyze point accuracy and assess dependability of survey for geodetic and surveying applications by using RTK, PPK, PPP and GCP outputs in UAV based photogrammetry. By performing different GNSS based positioning approaches, it is aimed to eliminate dependency of GCP's on the UAV based photogrammetric mapping. Elimination of GCP dependency benefits especially in studies that are carried out in disasters areas, hazardous mine and industrial areas.

Within this scope, 17 ground control points were used in ITU-Ayazağa Campus, size of study area is 455,000 m². Points were used as a GCP and test points. Every points were measured by relative static GNSS methods to obtain under 1 cm accuracy. As a result of evaluation, accuracy of horizontal position is found out under the 0.3 cm and vertical position accuracy around the 0.6 cm. ISTA (IGS) and PALA (ISKI UKBS) stations were reference stations in process.

Around the 950 images were gathered by multirotor DJI Phantom 4 RTK UAV in situ. Multirotor UAVs are more appropriate in small areas in terms of take-off area. Flight altitude was 80 meters from ground. Therefore, it provides 2 cm pixel resolution on images. Center of signaled GCP is painted with 2 cm diameter. Each image coordinates correction is obtained from local GNSS network called ISKI UKBS with N-RTK technique.

In order to evaluate RTK, PPK and PPP measurements on UAV based photogrammetry, Agisoft Metashape, commercial software, is executed. 7 different Orthophoto were evaluated with RTK, PPK, PPP and GCPs measurement and their combinations for comparison. Obtained comparisons and results are provided in the conclusions. Also, illustrations are added on Appendix.

İNSANSIZ HAVA ARACI İLE FOTOGRAMETRİDE UYDU KONUMLAMA TEKNİKLERİNİN KARŞILAŞTIRILMASI

ÖZET

İlk örneklerinin 20. yüzyılın başlarında görülmesine rağmen, İnsansız Hava Araçları'nın (İHA) askeri amaçlar dışında gündelik hayatın her alanında yaygın olarak kullanılmaya başlanması yüzyılın sonunu bulmuştur. Üretim maliyetlerinin düşmesi, farklı amaçlara uygun seçeneklerin artması bu süreci hızlandırmıştır. Ayrıca farklı ekipmanların kolaylıkla sisteme dahil edilmesi sayesinde oldukça geniş bir kullanıcı kitlesine kavuşmuştur. Askeri ve sivil amaçlı olarak İHA, pilotsuz olarak, kullanıcılar tarafından belirlenmiş görevleri yerine getiren ve yük taşıma kapasitesine sahip olan, uzaktan kumandalı veya otonom olarak uçabilen bir araçtır. Günümüzde kullanılan, insan ve yüksek hacimli yük taşıyabilen hava araçları (Uçak, zeplin vb.) ile İHA karşılaştırıldığında, maliyetlerin düşürülmesi, özel olarak inşa edilmiş kalkış pistlerine ihtiyacı olmaması ve boyutları sebebiyle özel görevlerde kullanılmak için İHA daha fazla tercih edilmektedir. Bu nedenle, tarım, ormancılık, maden, inşaat, arkeoloji, mimari gibi alanlarda yaygın olarak kullanılmaktadır. Öte yandan, kamera taşıma kabiliyeti, görüntüleme ve fotogrametrik tekniklerde kullanılmasına olanak sağlamaktadır. Geniş, uzak kırsal ve dağlık bölgeleri haritalamak ve modellemek için yaygın olarak kullanılan yüksek hacimli hava araçları, işçilik ve yer hizmetleri maliyetlerini artıran faktörlerdir. Bu nedenle belirli görevlerde, klasik fotogrametride kullanılan büyük boyutlu araçlar yerine İHA tercih etmek maliyetleri düşürmektedir.

Fotogrametrinin ana problemi, fotoğraflardan elde edilen iki boyutlu konum verisi ile gerçek küresel koordinatlar arasındaki ilişkiyi gösteren matematiksel modeli oluşturan eşdoğrultu ve eş düzlemsellik denklemlerinin bilinmeyen parametrelerini tanımlamaktır. Bu bilinmeyenler iç ve dış yöneltme parametreleri olarak tanımlanmaktadır. İç yöneltme elemanları odak uzaklığı, distorsiyon parametreleri ve fotoğraf izdüşüm koordinatlarıdır. Dış yöneltme elemanları ise fotoğraf çekme anındaki iz düşüm merkezinin küresel konumu ve resmin üç ekseninde dönüklük parametreleridir. Fotogrametrik kameraların iç parametreleri genellikle bilinmektedir. Klasik hava fotogrametrisinde, dış yönlendirme parametreleri, yeryüzünde tesis edilen ve hassas olarak ölçülen, yer kontrol noktalarının (YKN) yardımıyla hava triangülasyonu ile hesaplanır. YKN'lerin geniş alanlarda tesis edilmesi zaman alıcı ve maliyetlidir. Bu nedenle, hava triangülasyonunda ışın demetleri blok dengelemesi kullanılması sayesinde YKN'nin sayısı düşürülebilir.

Sensör sistemlerinin geliştirilmesi ve entegrasyonu ile İHA'lar, Küresel Uydu Konumlama Sistemleri (GNSS - Global Navigation Satellite Systems) ve İç Ölçme Birimi (IMU - Inertial Measurement Unit) 'den oluşan navigasyon sistemi sayesinde yönlendirilebilmektedir. GNSS ve IMU'nun İHA ile entegrasyonu hava fotogrametrisinde yeni yaklaşımlar ortaya çıkarmıştır. Görüntülerin yöneltmesinde kullanılan konum verilerinin ve dönüklük parametrelerinin eş zamanlı olarak elde edilmesi işlem yükünü azaltmaktadır. Dış yöneltme parametrelerinin sensörler sayesinde ölçülmesi yapılan çalışmaların çok daha az maliyetli yapılmasını sağlar.

Ayrıca yaklaşık dış yöneltme elemanlarının Hava Triangülasyonunda kullanılması, hesaplanacak noktaların doğruluğunu arttırmaktadır. GNSS verileriyle fotoğraf konumlarının belirlenmesi “Doğrudan Konumlandırma” (Direct Georeferencing) olarak adlandırılır. Ancak, mutlak GNSS konumlama tekniğinin doğruluğu, güvenilir sonuçlar almak için yeterli değildir. Öte yandan, GNSS ve IMU tarafından sağlanan başlangıç dış yöneltme değerleri, ışın demeti blok dengelemesinin büyük ölçüde doğruluğunu artırır. GNSS-IMU sensörleri sayesinde, daha az yer noktası ile sistemin doğruluğu artırılabilir.

Her ne kadar tek bir uydu alıcısının gözlem yaptığı mutlak GNSS konumlama tekniği düşük doğrulukta olsa da, ölçme doğruluğunu etkileyen iyonosferik gecikmenin modellenmesi ve uydu yörünge ve saat verilerinin yüksek doğruluklu olarak Uluslararası GNSS Servis (IGS- International GNSS Service) tarafından belirlenmesi sayesinde, mutlak konumlamanın doğruluğu cm mertebesinde hesaplanabilmektedir. Bu yöntem Hassas Konum Belirleme (PPP- Precise Point Positioning) olarak adlandırılmaktadır.

En az iki GNSS alıcısı ile eş zamanlı olarak yapılan gözlemlerle bağıl GNSS konumlama tekniğinin doğruluğu mm'ye kadar artar. Kullanılan GNSS alıcılarından en az biri konumu yüksek doğruluklu olarak bilinen ve ölçme süresince sürekli gözlem yapan referans istasyonu olarak seçilir; diğer GNSS alıcısı ise gezici olarak adlandırılır. Referans istasyonu ile diğer alıcılar arasındaki baz, GNSS ölçmelerinin dengelenmesi ile belirlenir. Bağıl Statik, Gerçek Zamanlı Kinematik (RTK- Real Time Kinematic) ve Post Process Kinematic (PPK) yöntemleri kullanılarak bağıl GNSS konumlandırma gerçekleştirilir. Eğer eş zamanlı olarak düzeltmeler ağ üzerinde gezici alıcıya aktarılıyor ise RTK (Real Time Kinematic) olarak, bazlar daha sonra epok epok çözülüyor ise PPK olarak adlandırılır. Eş zamanlı olarak düzeltme olarak konum belirlemek, ofisteki konum hesaplama yükünü ortadan kaldırmaktadır. Bu nedenle RTK ve PPK yöntemlerinin İHA'lara dayalı fotogrametri ile entegrasyonunun model doğruluğuna etkisi bu çalışmanın önemli bir çıktısı olacaktır.

Bu çalışmada; N-RTK, PPK, PPP ve YKN verileri kullanılarak oluşturulan modellerden elde edilen noktaların doğruluğunu analiz etmek ve jeodezik uygulamalar için GNSS konumlama tekniklerinin güvenilirliğinin değerlendirilmesi amaçlanmıştır. Bu kapsamda farklı yaklaşımlar uygulanarak, YKN'lerin fotogrametrik yöntemdeki bağımlılığı test edilmiştir. YKN bağımlılığının ortadan kaldırılması dağlık bölgelerin, afet bölgelerinin, tehlikeli maden sahalarının ve karmaşık sanayi bölgelerinin gözlemlenmesini kolaylaştıracaktır.

Bu kapsamda İTÜ Ayazağa Yerleşkesi'nde, çalışma alanı yaklaşık 455.000 m² olan bölgede 17 YKN, test ve kontrol noktası olarak kullanılmak üzere belirlenmiştir. Alanın genişliği göz önünde alınarak 17 noktanın yeterli olduğuna karar verilmiştir. Yatayda ve düşeyde 1 santimetrenin altında doğruluk elde etmek için, bütün YKN'nin konumu bağıl statik GNSS yöntemi ile belirlenmiştir. Değerlendirme sonucunda, yatay konum doğruluğunun ortalama 3 mm ve düşey konum doğruluğunun ortalama 6 mm olduğu tespit edilmiştir. İSTA (IGS) ve PALA (ISKI UKBS) istasyonları yakınlıkları ve güvenilirlikleri sebebiyle baz çözümleri için referans olarak alınmıştır. Statik ölçmeler Trimble Business Center ticari yazılımında değerlendirilmiştir.

Bu çalışmada fotogrametrik ölçme sürecinde multirotor DJI Phantom 4 RTK İHA kullanılmıştır. DJI Phantom 4 RTK çift frekanslı GNSS alıcısına, CMOS 1 adlı kamera ve IMU donanımlarına sahiptir. Multirotor İHA'lar, dikey kalkış yaptıkları için özel pistlere gereksinim duymamaları nedeniyle tercih edilmektedirler. Elde

edilen görüntülerde 2 cm piksel çözünürlüğüne ulaşmak için, uçuş yüksekliği yerden yaklaşık 80 metre olacak şekilde ayarlanmıştır. Ayrıca önceden tesis edilen YKN'lerinin merkezleri 2 cm çapında boyanmıştır. Uçuşta yaklaşık 950 adet fotoğraf çekilmiştir.

RTK methodu ile uçuş anında, resim çekme merkezi koordinatlarının yüksek doğrulukla belirlenmesi için ölçme düzeltmeleri internet bağlantısı ile ISKI-UKBS ağından alınmıştır. ISKI-UKBS ağına yatayda ± 2 , düşeyde ise ± 5 cm doğruluk sağladığı belirtilmektedir. N-RTK methoduyla alınan resim çekme merkezi koordinatları daha sonra ortofoto üretiminde kullanılmıştır.

Uçuş anında N-RTK yöntemi ile konumlandırma yapılırken aynı zamanda 0.2 saniyelik zaman aralığıyla alıcı ham GNSS ölçüleri kayıt edilmiştir. Daha sonra sürekli gözlem yapan IGS-ISTA istasyonunun aynı zaman dilimine ait 1 saniye zaman aralıklı GNSS gözlemleri temin edilmiştir. Bu gözlemler sayesinde, PPK yöntemi ile iki alıcı arasındaki bazı çözümleri yapılmış; önce İHA alıcı anteni ve daha sonra resim çekme merkezi koordinatları elde edilmiştir.

Fotoğrafik değerlendirme için otomatik değerlendirme yapan ticari AgiSoft Metashape Professional yazılımı kullanılmıştır. Bu yazılım fotogrametrik değerlendirme sürecinde Structure from Motion (SfM) yaklaşımını uygulamaktadır. Bu yaklaşımda koordinatlı veya koordinatsız bütün fotoğraflarda eşleşmiş noktalar yardımıyla 3 boyutlu yüzey oluşturulmaktadır. Agisoft Metashape Professional yazılımı ile N-RTK, PPK, PPP ve YKN, konum verileri kullanılarak 7 farklı ortofoto üretilmiştir. 17 YKN noktasından 12 nokta test noktası olarak seçilmiş, diğer noktalar ile hava triangülasyonu ile dijital ortamda ortofoto üretilmiştir. Aynı zamanda RTK, PPK ve PPP verileri ile 5 YKN noktasının kullanılması ile aynı bölgeye ait ortofotolar üretilmiştir.

Bağıl statik GNSS yöntemi ile elde edilen 17 YKN'nın konum verileri ile , ayrı ayrı RTK, PPK ve PPP verileri ile üretilen ortofoto üzerinden alınan YKN konum verileri karşılaştırılmış; yatay da yaklaşık benzer değerler yakalanırken düşeyde PPK ile PPP ile üretilen modellerin daha doğru olduğu görülmüştür. 5 YKN noktası ile üretilen ortofotolardan elde edilen 12 adet test noktasının konum verileri ile gerçek konumları arasındaki yatay farklar 2 cm'nin altında düşey farklar ise ortalama 2.5 cm olarak hesaplanmıştır. Üretilen bütün modellerde yatayda aynı farklar gözlenirken, düşeyde en iyi sonuçlar PPP ile YKN birlikte kullanılarak üretilen ortofotodan elde edilmiştir.

Bu sonuçlar sayesinde, istenilen konum doğruluğuna göre YKN tesis edilme gerekliliği, RTK, PPK veya PPP methodları kullanılarak ortadan kaldırılabilir. Aynı zamanda PPK ve PPP çıktılarından üretilen ortofoto verilerinin, RTK çıktılarından üretilen ortofoto verilerinden daha iyi olmasının sebebi olarak RTK ağına sağladığı konum doğruluğu olduğu düşünülmektedir. Bu durumdan kurtulmak için single-RTK tekniğinin kullanılması önerilir.

Tüm koordinat karşılaştırmaları ve istatistiksel sonuçlar, tezin sonuç kısmında irdelenmiş, çıkarımlar ve öneriler verilmiştir. Tüm çalışmaya ait ayrıntılı çizimler ve tablolar Ekler kısmına eklenmiştir



1. INTRODUCTION

UAVs (Unmanned Aerial Vehicle) have recently been preferred for geomatics engineering applications such as urban modeling, restoration, and digital terrain model generation. As an alternative platform for obtaining 3D point cloud, UAV is preferred in geodetic studies due to increased position accuracy with RTK (Real-Time Kinematic), PPK (Post-Processed Kinematic) and PPP (Precise Point Positioning). Its low cost also increases its preferability.

In traditional photogrammetry, georeferencing is mandatory process to transform real world coordinates with GCP (Ground control point) (Fazeli, Samadzadegan, & Dadrasjavan, 2016). However, the GCP collection which is known as indirect georeferencing is a time consuming and costly process. The alternative strategy is direct georeferencing method which includes 3D coordinate information obtained from on-board sensors mounted UAV (Gabrlik, 2015). In other words, direct-georeferencing method provides global coordinates and axis rotations using GNSS-IMU integration. Among the GNSS methods RTK, PPK and PPP techniques are the high precise positioning techniques. Those methods enhance point accuracy up to cm through GNSS methods (Subirana et al, 2013; Chudley et al, 2018; Zang et al, 2019).

RTK and PPK methods are included in the relative GNSS methods. If the corrections are simultaneously transmitted to the roaming receiver on the network, it is called RTK and if the baseline processed epoch by epoch later, it is called PPK. Processing data in the office enables to eliminate the blunders and noises on the position calculation. On the other side, by using Ionosphere-free combination and precise satellite orbit and clock data, GNSS positions are obtained more accurate with PPP. Therefore, it is expected that the integration of RTK, PPK and PPP methods with UAV based photogrammetry can increase the accuracy of the models.

Chen et al. (2018), compared two periods (October 2013 – July 2014) of UAV images with 150 m flight height on the beach of Wujiao Bay to monitor beach topography and its change. 10 GCPs in first period and 9 GCPs in second period were measured with

N-RTK method, and respectively 73 and 110 test point measured. Horizontal accuracy and vertical accuracy were estimated around the ± 7.7 cm and ± 11 cm respectively in both sessions. They defined that estimated position accuracies extracted from UAV based images were enough to monitor coastal change.

Chudley et al. (2019), acquired aerial images with stable wing UAV on Store Glacier, Greenland. They did not use any GCPs because of inaccessible and hazardous field. Instead of GCPs based orientation, it was decided to apply GNSS-supported aerial triangulation. During the UAV flight whose altitude was 450 m from ground, the location of photo acquisitions were determined using GNSS based kinematic relative positioning. It was calculated ± 12 cm position accuracy in horizontal and ± 14 cm position accuracy in vertical. They recommended to use GNSS aided UAV based photogrammetry for dangerous areas.

Zang et al. (2019), evaluated potential of UAV based photogrammetry for surface change detection using PPK technique. According the study, camera locations measured by PPK provided in cm level accuracy which is very close to GCPs results.

In this study, it was aimed to analyze and compare the accuracy of the points obtained from the created models, using RTK, PPK, PPP and GCP data and to evaluate the reliability of the methods for geodetic and photogrammetric applications. In this context, by applying different approaches, it is aimed to eliminate the dependence of GCPs in the photogrammetric method. In the process of photogrammetric evaluation, Structure from Motion (SfM) approach was applied. In this approach, 3 dimensional surface is created with the help of matched dots in all photographs with or without coordinates. Agisoft Metashape Professional software uses RTK, PPK, PPP and GCP position data to produce different orthophotos. Among the 17 ground control points, 12 points were selected as test points and orthophotos were produced digitally by air triangulation with other points. At the same time, using the combination between GCPs, and RTK, PPK and PPP data, 7 different orthophotos were produced for the same region.

Within this scope, 17 ground points were set in ITU-Ayazağa Campus, size of study area is around the 455,000 m². Ground points were set up to use as a control and test points. Every points were measured by relative static GNSS methods to obtain under 1 cm accuracy. As a result of evaluation, accuracy of horizontal position was found

out under 0.5 cm and vertical position accuracy around the 0.5 cm. In GNSS baseline processing of GCPs ISTA (IGS) and PALA (ISKI UKBS) continuous stations were used as reference stations.

In order to evaluate all satellite positioning methods outputs, Agisoft Metashape, commercial photogrammetric evaluation program, is executed. All methods outputs are used individually to evaluate images. On the other hand, in order to compare results and see effects of GCPs on RTK, PPK and PPP surveys, combination of GCPs and satellite positioning techniques are evaluated on photogrammetric software. Estimation with combined GCP data and direct-georeferencing data is called hybrid bundle adjustment.

Whole point coordinates extracted from models are compared with relative static GNSS measurements. Comparisons and results are provided in case study. Required tables, figures and illustrations are added on appendix. In last section, it is summarized all the arguments and recommendations based on interpretation of study outputs.

Although it has some vulnerabilities, using UAV in photogrammetric flight instead of full sized manned vehicle can reduce costs and save time for special missions. Integration of GNSS-IMU on the UAV based photogrammetry is providing high position accuracy depending on GNSS technique. Result of study approves reliability of the RTK, PPK and PPP techniques on UAV based photogrammetry.



2. COORDINATE SYSTEM IN GEODESY AND GLOBAL NAVIGATION SATELLITE SYSTEM

Determining position and its accuracy is one of the main tasks in geodesy. It is required to give a reference in order to define position of object in space. Defining of coordinate system with its origin and axes directions is necessary for positioning. Appropriate coordinates for geodesy and related fields are cartesian or curvilinear ellipsoidal due to physical shape of Earth.

Selecting best fitting ellipsoid for geoid is not sufficient to establish system. That's why, datum parameters define link between reference ellipsoid and geoid. Realization of terrestrial coordinate systems is called as reference frame and it is realized with geodetic reference network (Jekeli, 2006). On the other hand, issuing different datums requires transformation between them. The defined mathematical equations are used in order to carry out the transformation processes.

Beside the description of coordinate systems, determination of point positions with high accuracy require advance surveying techniques. Satellite based positioning (GNSS) technique is widely used for geodetic positioning purposes. Although it is effected by many error sources, errors might be eliminated by using different techniques and equipments.

2.1 Coordinate Systems

For a long time in development of civilization, celestial sphere of stars was most reliable reference to define location and navigation. It is still used in many geodetic applications. While determining the location of any point in space, coordinate systems are defined. Coordinates of any point can be explained by polar coordinate (λ, Θ, r) or cartesian coordinate (x, y, z) (Figure 2 1).

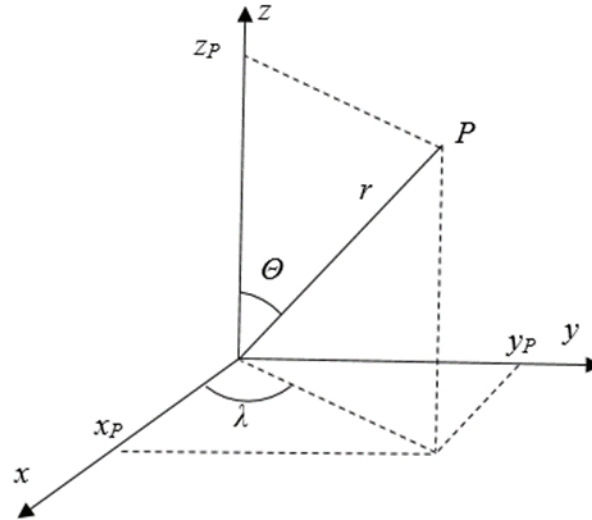


Figure 2.1 : Polar and Cartesian coordinates.

Polar coordinates consist of angles from x axis (λ) and z axis (θ) to direction of P point and, distance from origin (r). Also, distances along the x , y and z axis set off Cartesian coordinates (x_P, y_P, z_P).

Coordinate systems are described by definition of origin and orientation of axes directions. Because of heterogenic structure and shape of earth, it is required to identify special coordinate system which fits earth physical shape. Earth physical shape is accepted as geoid which is equipotential surface of earth gravity field that approximately fits global sea level (Hofmann-Wellenhof & Moritz, 2006). It is hard to mathematically modelling geoid due to physical shape and irregularities. By considering dimension and shape of geoid, ellipsoid which formed by rotation around the semi-minor axis, is best fitting closed surface for geoid. It can be mathematically modelled and formulated.

Nowadays it is widely utilized ellipsoidal coordinate system in geodetic application like satellite geodesy (Seeber, 2008). Rotational ellipsoid center are localized on the earth gravity center. Geodetic ellipsoidal coordinate system is defined as;

- X axis is located on intersection of equatorial plane and Greenwich meridian,
- Z axis and semi-minor axis are overlapped
- Y axis is completing a right handed system.

Figure 2.2 illustrates both geodetic cartesian and curvilinear coordinates.

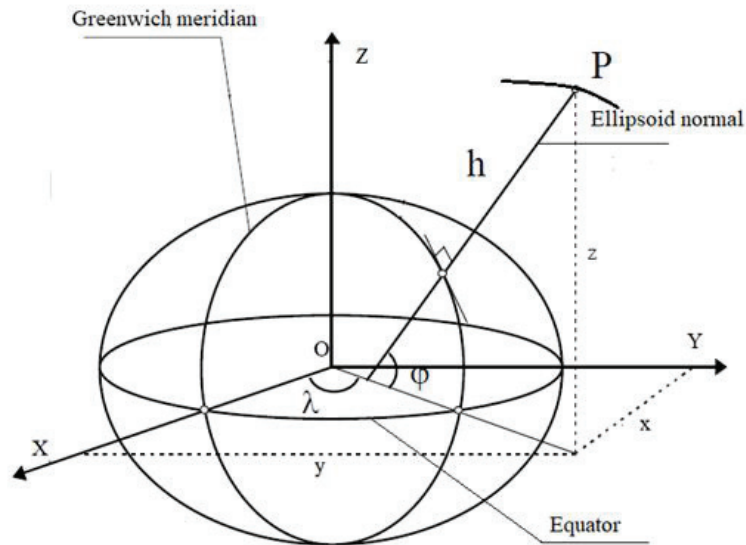


Figure 2.2 : Geodetic (Ellipsoidal) Curvilinear and Cartesian coordinates

Geodetic ellipsoidal coordinates are described as;

- ϕ is the ellipsoidal latitude and is described as angular distance from equatorial plane until ellipsoid normal in the meridian plane of point,
- λ is the ellipsoidal longitude that is anti-clockwise angular distance from Greenwich meridian plane until the meridian plane of the point measured on the equatorial plane,
- h is the ellipsoidal height measured from the surface of ellipsoid along the ellipsoid normal.

Geodetic cartesian and curvilinear coordinates; Figure 2.3 below illustrates relation between geodetic Cartesian coordinates and curvilinear coordinates which consist of (X, Y, Z) and (ϕ, λ, h) . In figure N represents the radius of curvature in the prime vertical and a and b respectively represent semi-major and semi-minor axis of ellipsoid.

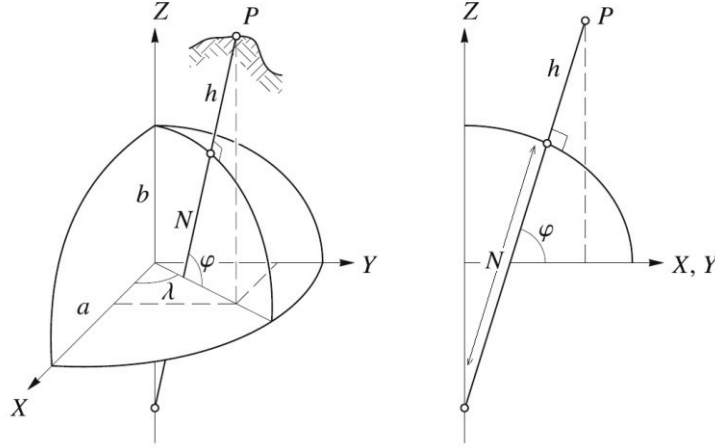


Figure 2.3 : Geodetic cartesian coordinates and curvilinear coordinates (Hofmann - Wellenhof, Lichtenegger & Wasle, 2007)

Equations between Cartesian and ellipsoidal coordinates are given by

$$\begin{bmatrix} X \\ Y \\ Z \end{bmatrix} = \begin{bmatrix} (N + h) \cos \varphi \cos \lambda \\ (N + h) \cos \varphi \sin \lambda \\ \left(\frac{b^2}{a^2} N + h \right) \sin \varphi \end{bmatrix} \quad (2.1)$$

Where N, radius of curvature, is calculated by

$$N = \frac{a^2}{\sqrt{a^2 \cos^2 \varphi + b^2 \sin^2 \varphi}} \quad (2.2)$$

Global geodetic and local geodetic coordinates; geodetic curvilinear or cartesian coordinates describe point locations globally. In applications, local coordinates are also used frequently. Origin of the local system is on the ellipsoid normal; u axis is oriented along the tangent surface normal, n axis is oriented to north along the meridian and e axis is oriented to east (Figure 2.5).

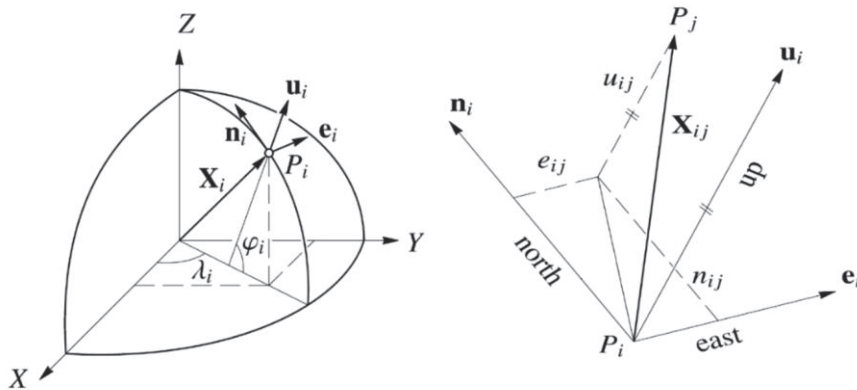


Figure 2.4 : Global geodetic and Local geodetic coordinates (Hofmann – Wellenhof et al, 2007)

$\mathbf{X}_i$ and $\mathbf{X}_j$ represent two coordinate vectors of terrestrial points P_i and P_j , in global system instead of using the component X, Y, Z . Baseline between $\mathbf{X}_i$ and $\mathbf{X}_j$ vectors is defined in global coordinate system by $\mathbf{X}_{ij} = \mathbf{X}_j - \mathbf{X}_i$. Moreover, $\mathbf{X}_{ij}$ vector can be described in local geodetic coordinates by the notation $\mathbf{x}_{ij} (n_j, e_j, u_j)$. With the help of n_i, e_i and u_i vectors (Hofmann – Wellenhof et al, 2007)

$$n_i = \begin{bmatrix} -\sin \varphi_i \cos \lambda_i \\ -\sin \varphi_i \sin \lambda_i \\ \cos \varphi_i \end{bmatrix}, \quad e_i = \begin{bmatrix} -\sin \lambda_i \\ \cos \lambda_i \\ 0 \end{bmatrix}, \quad u_i = \begin{bmatrix} \cos \varphi_i \cos \lambda_i \\ \cos \varphi_i \sin \lambda_i \\ \sin \varphi_i \end{bmatrix} \quad (2.3)$$

$\mathbf{x}_{ij}$ vector components in local system (n_j, e_j, u_j) is defined with $\mathbf{X}_{ij}$ by

$$x_{ij} = \begin{bmatrix} n_j \\ e_j \\ u_j \end{bmatrix} = \begin{bmatrix} n_i^T \\ e_i^T \\ u_i^T \end{bmatrix} \begin{bmatrix} X_j - X_i \\ Y_j - Y_i \\ Z_j - Z_i \end{bmatrix} \quad (2.4)$$

If the local geodetic system vectors n_i, e_i, u_i are collected in a column matrix $\mathbf{R}_i$. With the help of the rotation matrix $\mathbf{R}_i$

$$\mathbf{R}_i = \begin{bmatrix} -\sin \varphi_i \cos \lambda_i & -\sin \lambda_i & \cos \varphi_i \cos \lambda_i \\ -\sin \varphi_i \sin \lambda_i & \cos \lambda_i & \cos \varphi_i \sin \lambda_i \\ \cos \varphi_i & 0 & \sin \varphi_i \end{bmatrix} \quad (2.5)$$

The local geodetic coordinates of P_j is obtained as.

$$x_{ij} = \mathbf{R}_i^T \mathbf{X}_{ij}. \quad (2.6)$$

World Geodetic System 84 (WGS 84) is an earth fixed- earth centered terrestrial reference system and geodetic datum. It is mainly developed by United States for geodetic reference system for Global Positioning System (GPS) applications. Its origin is located on earth's center of mass (Seeber, 2008). Also, axis of system characterized as;

Z-axis = The direction of the International Earth Rotation and Reference Systems Service (IERS) Reference Pole (IRP).

X-axis = Intersection of the IERS Reference Meridian (IRM) and the plane passing through the origin and normal to the Z-axis.

Y-axis = Completes a right-handed, Earth-Centered Earth-Fixed (ECEF) orthogonal coordinate system.

Table 2.1 : WGS 84 parameters (Seeber, 2008)

Parameter	Notation	Value
Semi-major Axis	a	6378137.0 meters
Flattening Factor of the Earth	1/f	298.257223563
Nominal Mean Angular Velocity of the Earth	ω	7292115 x 10 ⁻¹¹ radians/second
Geocentric Gravitational Constant	GM	3.986004418 x 10 ¹⁴ meter ³ /second ²

The International Terrestrial Reference Frame (ITRF) arises from set of points with their 3-dimensional cartesian coordinates which effectuate an ideal reference system, the International Terrestrial Reference System (ITRS). ITRF is realized by depending on Global Navigation Satellite System (GNSS) and Very Long Base Interferometry (VLBI) geodetic measurements (IERS, 2013). Widely used Geodetic Reference System 80 (GRS 80) ellipsoid is accepted as reference ellipsoid for ITRF. GRS 80 And WGS 84 ellipsoid parameters are given on Table 2.2

Table 2.2 : Reference ellipsoids parameters (Seeber, 2008)

Ellipsoid	Year	a (m) Semi-major Axis	b (m) Semi-minor Axis	Flattening
GRS 80	1980	6378137	6356752.3141	1/298.2572
WGS 84	1984	6378137	6356752.3142	1/298.257224

ITRF89, ITRF96, ITRF2000, ITRF2005, ITRF2008, ITRF2014 are some of the solutions released by IERS (2013). Station positions and velocities with their variance/covariance matrices are part of datum solutions. ITRF 96 datum was selected as a reference datum with 2005 epoch in Turkey (LSMSDPR, 2018).

2.2 Coordinate Transformations

Many countries and associations build their own geodetic datum in their geodetic infrastructure. Coordinate transformation are applied to transform the point coordinates from one system to another between coordinate systems context of geodesy.

2.2.1 3D Helmert similarity transformation

Similarity transformation includes three shifts, three rotations and scale parameter. In similarity transformation both system are orthogonal systems. After transformation,

the edges grow and shrink at the same rate and the absolute values of the angles remain constant. Defining position, a P point in two different coordinate systems are shown in Figure 2.5.

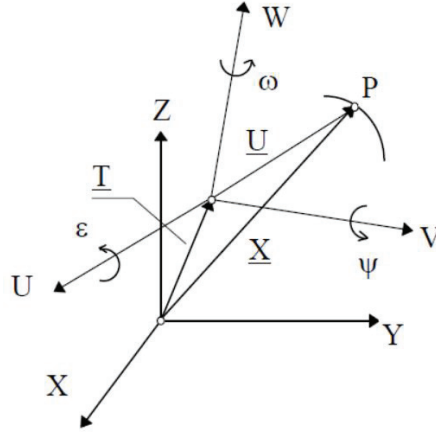


Figure 2.5 : 3D Similarity Transformation (Üstün, 1996)

X and U state different coordinate system and; $\underline{X}$ and $\underline{U}$ are coordinate vectors of P points in two systems. Furthermore, $\underline{T}$ vector shows U system's origin coordinates in X coordinate system. $\varepsilon, \psi, \omega$ are rotations around the U, V, W (Üstün, 1996). Equation 2.7 indicates the transformation between two systems,

$$\underline{X} = \underline{T} + (1 + \Delta)\underline{R}\underline{U} \quad (2.7)$$

where $(1+\Delta)$ is scale factor, $R(\varepsilon, \psi, \omega)$ vector is rotation matrix and $T(t_x, t_y, t_z)$ vector indicates translation matrix. R matrix consists of multiplication of rotation matrix around U, V, W axis. Rotations around the U, V, W axes, respectively are shown by:

$$\underline{R}_1(\varepsilon) = \begin{bmatrix} 1 & 0 & 0 \\ 0 & \cos \varepsilon & \sin \varepsilon \\ 0 & -\sin \varepsilon & \cos \varepsilon \end{bmatrix} \quad (2.8a)$$

$$\underline{R}_2(\psi) = \begin{bmatrix} \cos \psi & 0 & \sin \psi \\ 0 & 1 & 0 \\ -\sin \psi & 0 & \cos \psi \end{bmatrix} \quad (2.8b)$$

$$\underline{R}_3(\omega) = \begin{bmatrix} \cos \omega & \sin \omega & 0 \\ -\sin \omega & \cos \omega & 0 \\ 0 & 0 & 1 \end{bmatrix} \quad (2.8c)$$

matrix and multiplication respectively each other produce R rotation matrix.

$$\underline{R} = \begin{bmatrix} \cos \psi \cos \omega & \cos \varepsilon \sin \omega + \sin \varepsilon \sin \psi \cos \omega & \sin \varepsilon \sin \omega - \cos \varepsilon \sin \psi \cos \omega \\ -\cos \psi \sin \omega & \cos \varepsilon \cos \omega - \sin \varepsilon \sin \psi \sin \omega & \sin \varepsilon \cos \omega + \cos \varepsilon \sin \psi \sin \omega \\ \sin \psi & -\sin \varepsilon \cos \psi & \cos \varepsilon \cos \psi \end{bmatrix} \quad (2.9)$$

Since the translation, orientation and scale factors are not known at the beginning of the transformation process, the values should be determined with least squares adjustment. In both coordinate systems, a coordinate transformation is performed by taking advantage of three points which are present and whose coordinates are known.

2.2.2 Affine transformation

The main difference of Affine transformation than similarity transformation is the number of transformation parameters, In Affine transformation, there are 9 transformation parameters including 3 shift, 3 rotations and 3 scales. In similarity methods, the shape is preserved and the angles do not change. Affine transformation is commonly applied in photogrammetry because of heterogenic deformation on the axis. Transformation equation:

$$\underline{X} = \underline{T} + (1 + \Delta)\underline{R} \underline{U} \quad (2.10)$$

is same with other method. However, scale matrix is given in equation 2.11.

$$(1 + \Delta) = \begin{bmatrix} 1 + \Delta_1 & 0 & 0 \\ 0 & 1 + \Delta_2 & 0 \\ 0 & 0 & 1 + \Delta_3 \end{bmatrix} \quad (2.11)$$

2.3 Global Navigation Satellite System – GNSS

A GNSS contains a constellation of satellites around the Earth which continuously transmits signals that allow users to determine their three-dimensional location. Although for a long time only Global Position System (GPS) is fully operational, nowadays a number of global navigation satellite systems including GLONASS, BeiDou and Galileo are currently operational (Subirana et al, 2013).

The GNSS positioning principle depends on measuring a pseudoranges which involving the distances of a user to at least four GNSS satellites with known coordinates (Figure 2.6). Receivers receive the transmitted signals and navigation data by the satellites. Hence, the user location can be estimated to an accuracy of several meters. On the other hand, performing more advanced techniques can be resulted sub-centimeter level positioning accuracy (Subirana et al, 2013). The distance between the satellite and receiver is computed by designating the time difference between transmission and receiving of the signal. Mathematically and geometrically, it is required at least 3 satellite to define point position (lat, long and height or X, Y, Z).

On the other hand, there is not synchronization between the clock in the receiver and the clock in the satellite. This brings an additional unknown that is clock error. Hence at least 4 observations and 4 satellites are required to determine the location of a receiver. Therefore, the term pseudorange is preferred (Seeber, 2008).

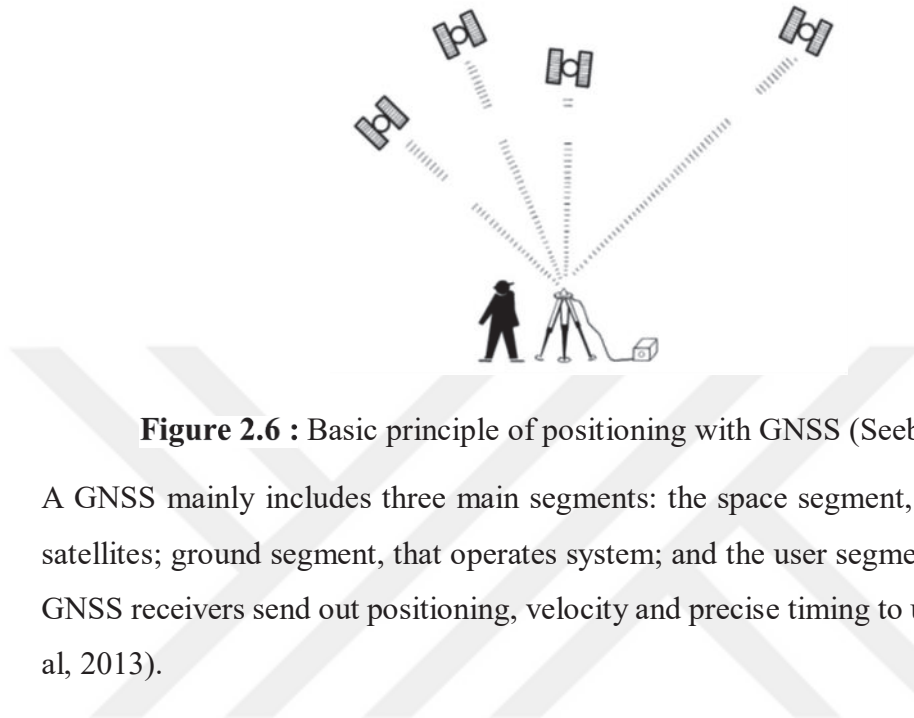


Figure 2.6 : Basic principle of positioning with GNSS (Seeber, 2008)

A GNSS mainly includes three main segments: the space segment, that contains the satellites; ground segment, that operates system; and the user segment, which has the GNSS receivers send out positioning, velocity and precise timing to users (Subirana et al, 2013).

Main GNSS signals L1 and L2 which cover two signal codes, are transmitted by the satellites frequency bands; the Coarse/Acquisition (C/A) code on L1 and the Precision (P) code on L1 and L2. GNSS receivers, which are enable to read both frequencies, could read the P-code. Basic receivers can read only the C/A-code that is basic GNSS signal. The advanced receivers can also read the carrier phase on which the GNSS signals are carried. These carrier phase measurements provide the high precision.

2.3.1 GNSS error sources

GNSS methods depend on measuring time differences between transmission and receiving of the signal spread by the satellites. Hence, the accuracy of the location relies on the accuracy of the measured time differences. The GNSS signals move forward in a line with speed of light. Because of ionosphere and troposphere, signal can be delayed or accelerated. Table 2.3 given below shows main error sources.

Table 2.3 : GNSS error budget (Seeber, 2008)

Error Source	RMS Range Error
<i>Satellite</i>	
– orbit	1 – 2 m
– clock	1 – 2 m
<i>Signal propagation</i>	
– ionosphere (2 frequencies)	cm – dm
– ionosphere (model, best)	1 – 2 m
– ionosphere (model, average)	5 – 10 m
– ionosphere (model, worst)	10 – 50 m
– troposphere (model)	dm
– multipath	1 – 2 m
<i>Receiver</i>	
– observation noise	0.2 – 1 m
– hardware delays	dm – m
– antenna phase center	mm – cm

2.3.1.1 Satellite orbits and clocks error

GNSS receiver receives the satellite orbit and time of transmission of the GNSS signal. The position of the satellite is evaluated for the receiver position computation. Nevertheless, both the satellite orbits and the satellite time transmitted in the signal including errors. Therefore, because of the errors in satellite orbit, the computed location for the receiver includes error. The delay of transmission introduces an error in the time difference estimation and hence in the range determination. Also, these errors affect the accuracy of the computed point coordinates (Hofmann – Wellenhof et al, 2007).

By the ground control segment, precise orbits and clock errors can be estimated. The precise orbit products are published by the International GNSS Service (IGS). However, these products are not published instantaneously. To increase accuracy, users can download these products from online servers after the measurements.

2.3.1.2 Ionospheric and tropospheric delays

GNSS signals travel around the 20000 km along the atmosphere, after transmissions. Because of context of ionosphere and troposphere GNSS signals are delayed. Therefore, range can be measured longer. By using dual frequency GNSS receiver and differential GNSS, atmospheric delay can be modelled.

On the other hand, troposphere contains water vapor, distribution of substance in there are heterogeneous. So, delay of signal depends weather. That is why modelling troposphere more difficult than ionosphere (Hofmann -Wellenhof et al, 2007).

2.3.1.3 Multipath effects

It occurs when GPS signals are propagated over two or more paths during the observation (Figure 2.7). This is one of the most important sources of errors. Errors caused by multipath cannot be corrected by differential approach, since they depend on the local reflection geometry around each receiver antenna. The easiest and most effective way to prevent this error is to ensure that there are no reflective surfaces near the receiver and, also special equipment decrease reflected signals. If a dual frequency receiver is performed, the effect of multipath error can be minimized.

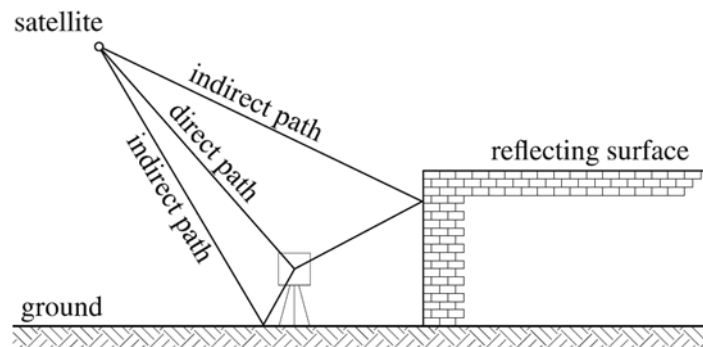


Figure 2.7 : Multipath Effect (Hofmann Wellenhof et al, 2007)

2.3.1.4 Receiver errors

The antenna of the GNSS receiver can be affected different kinds of signals like electromagnetic currents from power lines. These effects cause noise in signal context. Furthermore, antenna contains error sources. Mechanical center and the electronic center generally located at different place inside of the antenna. During observation the GNSS-receiver determine the electronic center of the antenna. The identification of the electronic center is another source for errors.

2.3.1.5 Satellite geometry

The distribution of the measured satellites in space also affects the coordinate calculations. If the observed satellites are very close to each other, the position accuracy to be obtained will be low and if the distribution is homogeneous, the position accuracy will be high. Geometric errors are defined as Geometric Dilution of Precision

(GDOP). Position (3D) Dilution of Precision (PDOP) refers to the distortion in the GDOP position, Vertical Dilution of Precision (VDOP), which denotes the distortion in the vertical position and TDOP (Time Dilution of Precision) which denotes the distortion in time (Hofmann -Wellenhof et al, 2007). The small values in the GDOP indicate that the distribution of satellites in space is good, in other words, the accuracy of position accuracy due to satellite geometry will be low.

2.3.2 GNSS positioning techniques

To obtain more accurate GNSS position, different measurement techniques are performed according to the desired sensitivity and purpose of GNSS measurements. Observation time, receiver type, location and number of satellites and measurement method effects obtained coordinate accuracy (El-Rabbany, 2002). Widely used code pseudorange P , and phase pseudorange Φ , equations are

$$P = \rho + cdt_r + cdt_b + \Delta_{trp} + \Delta_{ion} + \varepsilon \quad (2.12)$$

$$\Phi = \rho + cdt_r + cdt_b + \lambda N + \Delta_{trp} - \Delta_{ion} + \varepsilon \quad (2.13)$$

Where t is time, ρ is geometric range between satellite and receiver, cdt_r is satellite and receiver time error, cdt_b is the clock biases difference between satellite and receiver, c represents the speed of light, Δ_{trp} and Δ_{ion} represent respectively tropospheric and ionospheric delay, λ is the wavelength, N represents the phase ambiguity and ε represents other errors (Dawidowicz & Krzan, 2014).

Directly observing position of a single point (latitude, longitude, altitude or X, Y, Z) is called Absolute Point Positioning. Relative positioning is the determination of the positions of the points relative to other points. If the point location is stationary such as ground control point, it is stated as static location determination; if measurement platform is moving such as aircraft, and ships, it is called kinematic positioning. Real-time positioning can be best solution in order to navigate moving vehicles. Post-processing of the GNSS measurements can be carried out at a later time in order to obtain more accurate results by means of GNSS service providers.

2.3.2.1 Absolute positioning

During the observation, only one receiver is used to obtain signals in order to determine coordinates of the points by connecting at least 4 satellites (Hofmann -

Wellenhof et al, 2007). There are two approaches which differ from each other by modelling error budget (El-Rabbany, 2002).

Standard point positioning – SPP; as indicated before, one receiver observes at least four satellite or more in order to define its own position. While, the ranges are issued from C/A-code and P-code, the receiver obtains satellite location from navigation message. Measured ranges are affected by clock error, atmospheric delay and other errors owing to satellite and receiver. That is why its reliability is lower than other techniques.

Precise point positioning – PPP; while taking into account the point positioning methods, clock errors, orbit errors and atmospheric effects are main limitation for high accuracy positioning. Instead of the relative techniques, which requires measurements from reference station, in PPP technique, precise satellite orbits, clock corrections and, ionospheric and tropospheric models, solid earth and ocean tides, antenna phase-center offsets and variations, carrier-phase wind-up, relativistic effects etc are used in the post-processing (Rizos, Janssen, Roberts, & Grinter, 2012). Therefore, accurate satellite clock and orbit data provided by IGS, and dual frequency code-pseudoranges and carrier phase observations are used to improve the positioning accuracy (Hofmann-Wellenhof et al, 2007). The prior model depends on ionosphere-free combination of code pseudorange and carrier phases, which effectively remove ionospheric delay. Observation with dual frequency receiver enables to remove first order ionospheric delay up to 99.9% by combination of ionosphere-free combination (Eq. 2.11) (Subirana et al, 2013). Ionosphere free combination are described:

$$P_c = \frac{f_1^2 R_1 - f_2^2 R_2}{f_1^2 - f_2^2}, \quad \Phi_c = \frac{f_1^2 \Phi_1 - f_2^2 \Phi_2}{f_1^2 - f_2^2} \quad (2.14)$$

where R_1 and R_2 are ranges, Φ_1 and Φ_2 are phases and f_1 and f_2 are frequencies. By means of Global Ionosphere Maps promoted by IGS, ionospheric delays can be also eliminated. However receiver clock error, tropospheric delay and others are still unknown to be predicted along with the stations.

2.3.2.2 Relative positioning

GNSS relative positioning is based on two or more receivers, which observing same satellites to determine their relative coordinates. GNSS receiver, set up at a point which is known in coordinates, is referred as the reference station and maintains observing.

The other GNSS receiver is called the rover, moving around the area to measure the coordinates of different points. Instead of taking coordinates directly from the GNSS satellites, the baseline between the two receivers is calculated to avoid common error sources (Figure 2.8).

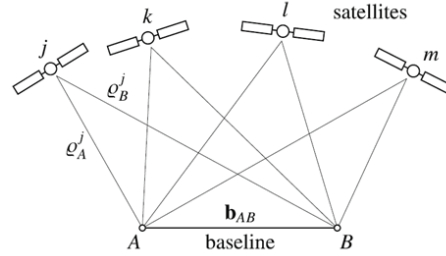


Figure 2.8 : Basic concept of relative positioning (Hofmann Wellenhof et al, 2007)

Relation between baseline vector $\mathbf{b}_{AB}$ and position vectors X_B and X_A are formulated as

$$X_B = X_A + \mathbf{b}_{AB} \quad (2.15)$$

and $\mathbf{b}_{AB}$ vector are

$$\mathbf{b}_{AB} = \begin{bmatrix} X_B - X_A \\ Y_B - Y_A \\ Z_B - Z_A \end{bmatrix} = \begin{bmatrix} \Delta X_{AB} \\ \Delta Y_{AB} \\ \Delta Z_{AB} \end{bmatrix} \quad (2.16)$$

Code measurements (Eq. 2.12) or phase measurements (Eq. 2.13) can be used during relative positioning. Both reference and rover points should observe simultaneously satellites. Therefore, signals and a particular satellites contains more or less identical errors and biases (El-Rabbany, 2002). Common error can be eliminated by estimating baseline between the reference and rover station depending on the distance between.

Static relative positioning; are performed postprocess to obtain point position with high accuracy. At least two or more stationary receiver observe same satellites. Reference station is set up on point whose coordinates are known very precisely. The other receiver is set up over the point, whose coordinates are not known. Furthermore, only one reference station can assist several receivers. Required observation time for high accuracy positioning is based on baseline length, and number of visible satellite and geometry.

Kinematic relative positioning; while reference receiver is stationary, remote receiver or receivers moving. They also observe at least 5 satellites simultaneously (Figure 2.9). A large number of points can be measured rapidly by means of kinematic technique. On the other hand, Real-Time Kinematic (RTK) technique, which also performs carrier-based ranging and detects ranges that are more accurate than through code-based positioning. The main purpose is decreasing and eliminating errors common to reference station and receiver pair by taking corrections in real time (El-Rabbany, 2002). RTK reference station should lay over a service area expending about 10 or 20 kilometers due to the rigours to determine phase ambiguities and spatial de-correlation of the errors (ESA, 2019). The rover receivers obtain reference receiver coordinates and measurements via communication (radio) link. Software loaded into rover receiver integrates and processes GNSS measurements and coordinates observed by both reference and rover receiver in order to estimates rover coordinates.

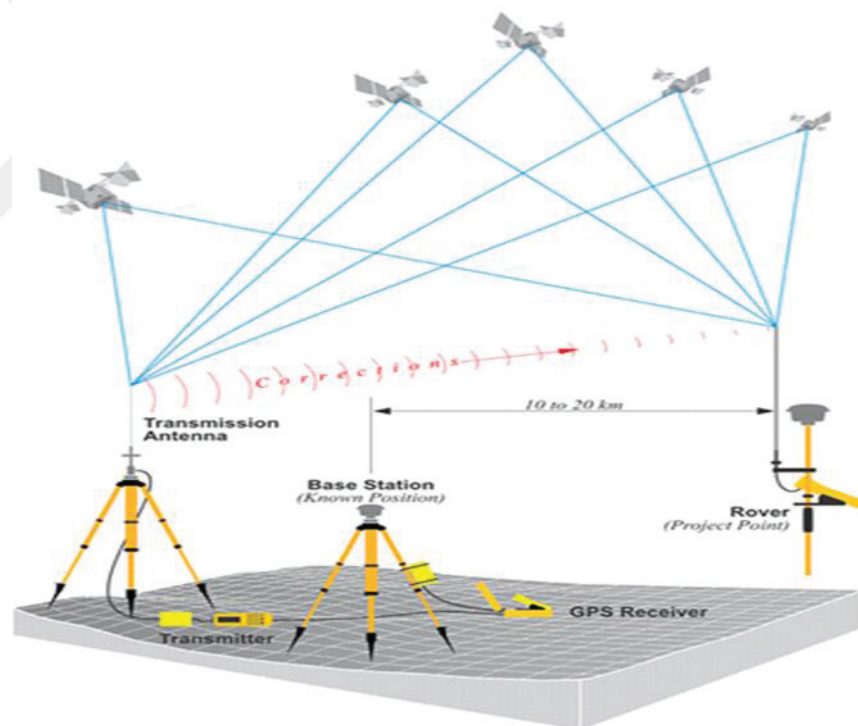


Figure 2.9 : Real Time Kinematic (URL2)

Owing to service range limitation, several permanent stations are set up to transmit data remote areas. Main goal behind the setting up RTK Continuously Operating Reference Stations Networks (CORS) is to deal with this limitation, to send out corrections by network of RTK stations (N-RTK) simultaneously (IAG, 2007).

Beside the RTK and N-RTK, Post-Processed Kinematic (PPK) is also another kinematic relative positioning technique. Instead of obtaining corrections in real time, PPK algorithms are performed in office to get accurate positions. There are several advantage of PPK because of having more flexibility in processing collected GNSS data (El-Rabbany, 2002). It eliminates radio link related problems like signal obstruction and limitation of coverage. Furthermore, error based reference and rover receiver can be removed in post-processed methods.



3. UAV BASED PHOTOGRAMMETRY

In last two decades, using UAVs in several applications from war industries to earth sciences is continuously increasing. Implementation of GNSS-IMU sensors on UAVs and developments of photogrammetric evaluation software provide accurate data for mapping. Relation between object space and image space is formed by collinearity equations. Obtaining mapping data from images is possible by conventional (aerial triangulation) and modern (structure from motion) photogrammetric approaches. On the other hand, assistance of navigational sensors is able to define exterior orientation parameters directly.

3.1 Brief Definition of UAV Based Photogrammetry

Originally “Photogrammetry” was formed by the three Greek words; phot (light), gramma (draw) and meter (measure). As technical term photogrammetry is measuring object coordinates or earth surface coordinates in 2 or 3 dimensions over the photographs. As the American Society for Photogrammetry and Remote Sensing (Wolf et al, 2014) explained that “*Photogrammetry is the art, science, and technology of obtaining reliable information about physical objects and the environment through processes of recording measuring and interpreting images and patterns of electromagnetic radiant energy and other phenomena.*” From the beginning of the photogrammetry many proto-methods used to achieve the best spatial model, but after the technological advancements, analog photogrammetry left its place to the digital photogrammetry, which have made much feasible to indicate object coordinates via using aerial photographs.

While aerial photogrammetry is performing for mapping, flight tracks should cover a spaces, which will bring photographs to overlay a common strip of ground. Covered space between flight strips, sidelap, is about 25% of the width of the area of photographs; besides, each images in the flight track lay overs an area that overlaps the area lay overs by previous image by about %60 (Figure 3.1) (Mikhail, Bethel & McGlone, 2001).

In the process of producing knowledge by photogrammetry, mathematical models are used to reach the most approximate coordinates through intersecting images. This model includes single image orientation and stereo image pair orientation that provide more accurate results while one aims to find precise coordinates of a place by using photographs. Recent developments show that Structure from Motion (SfM) as an approach has remarkable outcomes with evolving air triangulation and direct georeferencing methods by using mathematical models and advanced systems likewise GNSS and IMU. Eventually, photogrammetry has been sophisticated over the years, mostly in the aerial field with UAVs. Airborne photogrammetry widely used in order to mapping extensive earth surfaces by using photographs taken from aircraft, that is why this part contains UAV based photogrammetry.

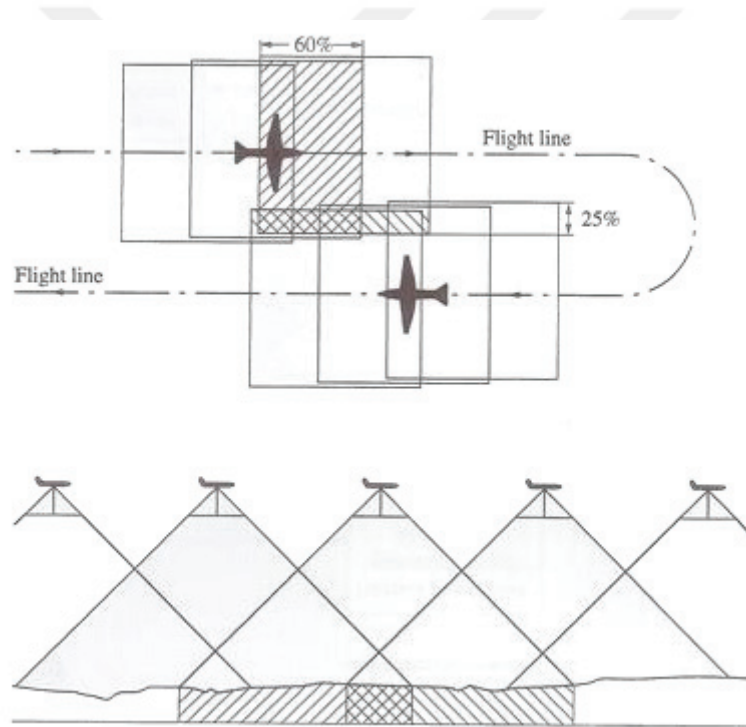


Figure 3.1 : Strip overlap (above) and overlap along the line (below) (Mikhail et al, 2001)

Along with the development of small size aircraft (Figure 3.2), UAV's have been started to used broadly as a part of airborne photogrammetric technique instead of big size manned air vehicles. UAV is defined as remotely controlled or autonomous aircraft has not pilot on-board. Unmanned Vehicle Systems (UVS) international categorizes UAV's three main groups; tactical, strategical UAVs, and special tasks UAVs (Remondino et al, 2012).



Figure 3.2 : Multi-rotor and stable wings UAVs

Basically, UAV consists of three main components; avionic system, payload transmitter and payload. Avionic system contains GNSS antenna and IMU, electronic and mechanical devices for flight. High resolution cameras and LIDAR are widely mantled on-board about photogrammetric purposes. Figure 3.3 shows main parts of UAV.

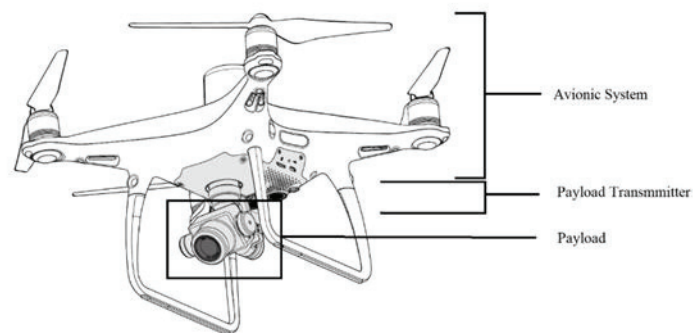


Figure 3.3 : Parts of UAV

3.2 Mathematical Model

A photograph consists of registration of the reflected rays through a single perspective center to a two-dimensional plane. It has a central projection geometry and it is a two-dimensional representation of three-dimensional space. The image point and the direction of the ray passing through the perspective center can be determined by the internal and external orientation parameters of single image. However, three-dimensional coordinates cannot be calculated. Three dimensional coordinates can be estimated by intersecting the rays passing through the perspective center of the same

terrain point in stereo image pairs, which overlapped each other (Kraus, 1992). It is necessary to know the coordinates of image and the point where the ray passing through the projection center intersects the earth to orient a single photograph. Hence, it requires height data of the area covered by the photograph on the earth.

The relation between the image coordinate system and the terrestrial coordinate system is shown in the Figure 3.4. The main idea behind the between two coordinate systems is that the projection center (L), the image point a and the corresponding field point A are on a single line (collinearity condition) (Kraus, 1992). This line correlate image coordinate system and the terrestrial coordinate system. The correlation between this image and the terrestrial coordinate systems can be expressed by interior and exterior orientation parameters.

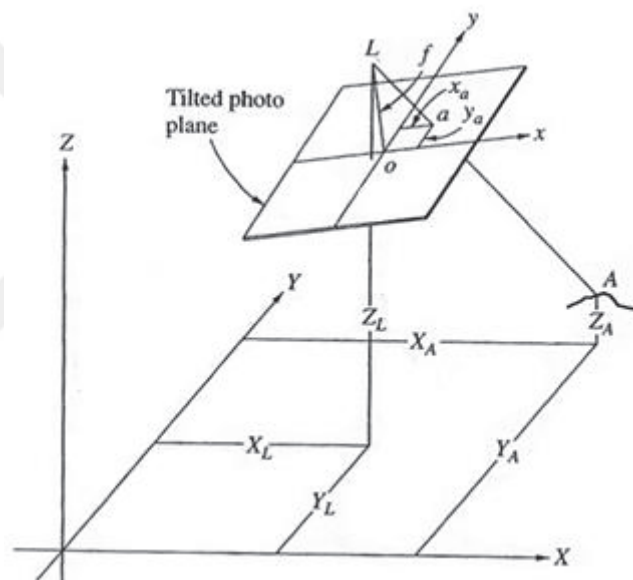


Figure 3.4 : Relation between image and terrestrial coordinate system.

In Figure 3.4;

- O is principle point,
- a is position of land point A on image
- x_a, y_a are coordinates of A on the image.

Interior parameters of camera are;

- f or c is focal length or camera constant
- x_0, y_0 is principal point coordinates on image frame

Exterior orientation parameters;

- X_L, Y_L, Z_L are coordinates of perspective center (at time of exposure)

- ω, ϕ, κ are rotation between image coordinate system and terrestrial coordinate system.

Collinearity equation are written by interior and exterior orientation parameters (Mikhail et al, 2001).

M (3 x 3) rotation matrix

$$\begin{bmatrix} x \\ y \\ z \end{bmatrix} = M \begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix} \quad (3.1)$$

Since the rotation matrix is orthogonal, the preceding formula can be expressed as follows.

$$\begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix} = M^T \begin{bmatrix} x \\ y \\ z \end{bmatrix} \quad (3.2)$$

In equations (3.1) and (3.2) given two coordinate systems are assumed to be the same origin and there is no difference in scale between them. If it is taken these factors into consideration and brought the main point coordinates correction to the image coordinates and set the camera constant as the z coordinate, the following formula (3.3) is obtained.

$$\begin{bmatrix} x_a - x_0 \\ y_a - y_0 \\ -c \end{bmatrix} = kM \begin{bmatrix} X_A - X_L \\ Y_A - Y_L \\ Z_A - Z_L \end{bmatrix} \quad (3.3)$$

If it is written with M matrix elements;

$$\begin{bmatrix} x_a - x_0 \\ y_a - y_0 \\ -c \end{bmatrix} = k \begin{bmatrix} m_{11} & m_{12} & m_{13} \\ m_{21} & m_{22} & m_{23} \\ m_{31} & m_{32} & m_{33} \end{bmatrix} \begin{bmatrix} X_A - X_L \\ Y_A - Y_L \\ Z_A - Z_L \end{bmatrix} \quad (3.4)$$

When it is performed the multiplication of the right side in the above equation, we get three scalar equations.

$$x_a - x_0 = k * [(X_A - X_L) * m_{11} + (Y_A - Y_L) * m_{12} - (Z_A - Z_L) * m_{13}] \quad (3.5)$$

$$y_a - y_0 = k * [(X_A - X_L) * m_{21} + (Y_A - Y_L) * m_{22} - (Z_A - Z_L) * m_{23}] \quad (3.6)$$

$$c = k * [(X_A - X_L) * m_{31} + (Y_A - Y_L) * m_{32} - (Z_A - Z_L) * m_{33}] \quad (3.7)$$

Equations (3.5) and (3.6), (3.7) by dividing the scale factor is eliminated and the following formulas are obtained. Central Projection Equations:

$$x_a = x_0 - c * \frac{[(X_A - X_L) * m_{11} + (Y_A - Y_L) * m_{12} - (Z_A - Z_L) * m_{13}]}{[(X_A - X_L) * m_{31} + (Y_A - Y_L) * m_{32} - (Z_A - Z_L) * m_{33}]} \quad (3.8)$$

$$y_a = y_0 - c * \frac{[(X_A - X_L) * m_{31} + (Y_A - Y_L) * m_{32} - (Z_A - Z_L) * m_{33}]}{[(X_A - X_L) * m_{31} + (Y_A - Y_L) * m_{32} - (Z_A - Z_L) * m_{33}]} \quad (3.9)$$

$$X_A = X_L + (Z_A - Z_L) * \frac{[(x_a - x_0) * m_{11} + (y_a - y_0) * m_{21} - (c) * m_{31}]}{[(x_a - x_0) * m_{13} + (y_a - y_0) * m_{23} - (c) * m_{33}]} \quad (3.10)$$

$$Y_A = Y_L + (Z_A - Z_L) * \frac{[(x_a - x_0) * m_{12} + (y_a - y_0) * m_{22} - (c) * m_{32}]}{[(x_a - x_0) * m_{13} + (y_a - y_0) * m_{23} - (c) * m_{33}]} \quad (3.11)$$

Equations (3.8) - (3.11) are called collinear equations or central projection equations.

If the M rotation matrix is rotated in three axes by the angles ω , ϕ , κ respectively;

$$M = M_\kappa * M_\phi * M_\omega \quad (3.12)$$

$$M_\omega = \begin{bmatrix} 1 & 0 & 0 \\ 0 & \cos \omega & \sin \omega \\ 0 & -\sin \omega & \cos \omega \end{bmatrix} \quad (3.13)$$

$$M_\phi = \begin{bmatrix} \cos \phi & 0 & \sin \phi \\ 0 & 1 & 0 \\ -\sin \phi & 0 & \cos \phi \end{bmatrix} \quad (3.14)$$

$$M_\kappa = \begin{bmatrix} \cos \kappa & \sin \kappa & 0 \\ -\sin \kappa & \cos \kappa & 0 \\ 0 & 0 & 1 \end{bmatrix} \quad (3.15)$$

Image orientation can be characterized in two terms as interior and exterior orientation. These parameters create both the relation between the projection center and the image frame (interior orientation); and, the relationship between the projection center and the terrain (exterior orientation).

3.2.1 Single image orientation

Both interior and exterior orientation parameters are represented on Figure 3.4 where it is seen that projection center (L), point position on image (a) and terrestrial point (A) are on the same line. However, without knowing height (Z) coordinate, the point A represented in the image as a point, can be anywhere on the ray passing through the O and a points (Mikhail et al, 2001). Therefore, in order to find the position of the point in the field, the height (Z) value of that point must be known. It is clearly seen that in order to solve the equations 3.10 and 3.11, Z height coordinates should be known beside the internal and external orientation parameters on the right side of the equations. Then, both X_A and Y_A coordinates can be calculated by solution of both equations.

3.2.2 Stereo-pair image orientation

When, the stereo pair images' exterior orientation parameters are known at the time of image acquisition; the ray on the left image formed by the projection center (L_1) and image point (a_1); and, the ray on the right image formed by the projection center (L_2) and image point (a_2) intersect at the point on the ground (Figure 3.5). They are called collinear or homolog rays. In this way, stereo pair of images, which internal and external orientation parameters known might be oriented.

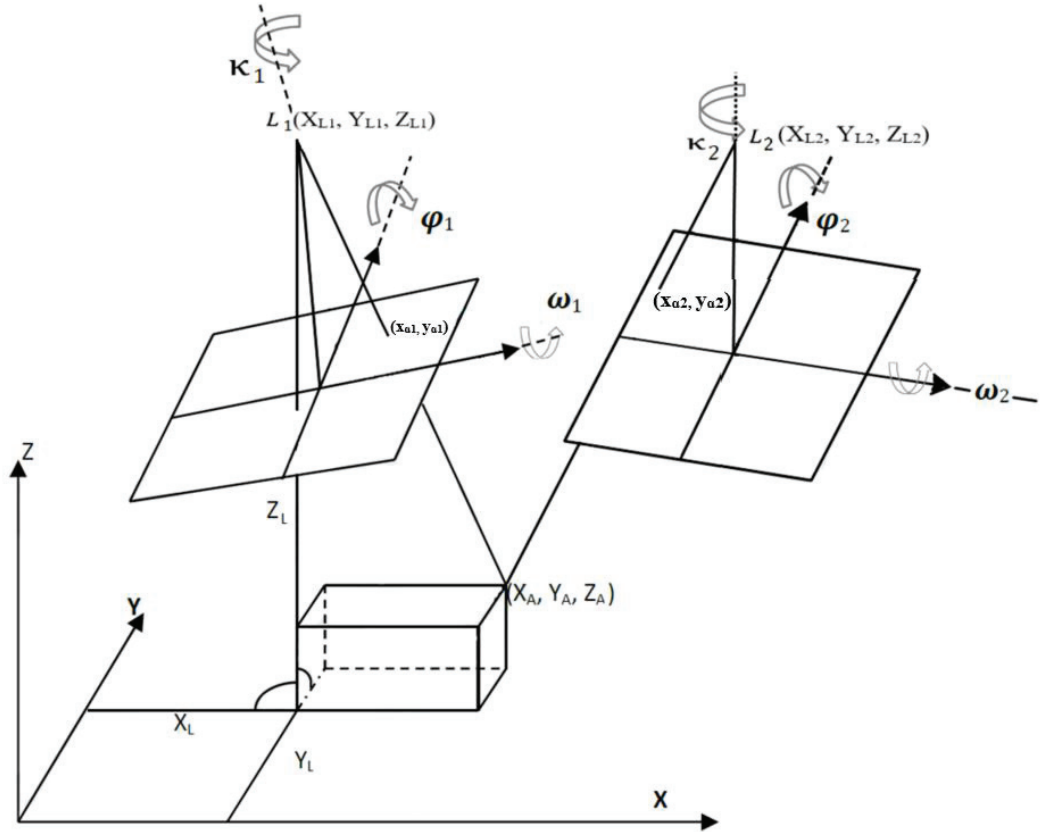


Figure 3.5 : Stereo Image orientation (Kraus, 1992)

Both image coordinates of A point should be measured on both images to evaluate stereo-pair images. If equations (3.10) and (3.11) are written for both images;

$$\frac{[(x_{a1}-x_0)*m_{11}+(y_{a1}-y_0)*m_{21}-(c)*m_{31}]}{[(x_{a1}-x_0)*m_{13}+(y_{a1}-y_0)*m_{23}-(c)*m_{33}]} = k_{x1} \quad (3.16)$$

$$\frac{[(x_{a1}-x_0)*m_{12}+(y_{a1}-y_0)*m_{22}-(c)*m_{32}]}{[(x_{a1}-x_0)*m_{13}+(y_{a1}-y_0)*m_{23}-(c)*m_{33}]} = k_{y1} \quad (3.17)$$

$$\frac{[(x_{a2}-x_0)*m_{11}+(y_{a2}-y_0)*m_{21}-(c)*m_{31}]}{[(x_{a2}-x_0)*m_{13}+(y_{a2}-y_0)*m_{23}-(c)*m_{33}]} = k_{x2} \quad (3.18)$$

$$\frac{[(x_{a2}-x_0)*m_{12}+(y_{a2}-y_0)*m_{22}-(c)*m_{32}]}{[(x_{a2}-x_0)*m_{13}+(y_{a2}-y_0)*m_{23}-(c)*m_{33}]} = k_{y2} \quad (3.19)$$

Image coordinates are obtaining as;

$$X_A = X_{L1} - (Z_A - Z_{L1}) * k_{x1} \quad (3.20)$$

$$Y_A = Y_{L1} - (Z_A - Z_{L1}) * k_{y1} \quad (3.21)$$

$$X_A = X_{L2} - (Z_A - Z_{L2}) * k_{x2} \quad (3.22)$$

$$Y_A = Y_{L2} - (Z_A - Z_{L2}) * k_{y2} \quad (3.23)$$

If equation (3.20) and (3.22) are estimated together;

$$Z_A = \frac{X_{L2} - Z_{L2}k_{x2} + Z_{L1}k_{x1} - X_{L1}}{k_{x1} - k_{x2}} \quad (3.24)$$

Z_A height coordinate can be estimated. Also; Z value can be estimated by equations (3.21 and 3.23). While processing, there will be four equations for 3 (X , Y , Z ,) coordinates on stereo pair images. Therefore, coordinates are estimated by the least square adjustment technique (Wolf et al, 2014; Kraus, 1992).

3.3 Structure from Motion (SfM)

As a photogrammetric approach, structure from motion called SfM exists since the end of the '70s (Ullman, 1979). It is used to produce 3D models by using overlapped images taken from different locations. Beside the geosciences, it is widely used in archaeology, architecture and agriculture. Not only ortho-rectified imagery but also dense point cloud can also be produced by this approach. Although it uses overlapping photographs, it differs from controversial photogrammetry by aspect of solving camera location, geometry of scene and orientations without requirement of GCPs which coordinates are known (Westoby, et al, 2012) and created orthomosaics by SfM can be referenced by small number of GCPs by using 3-D similarity transformation.

The photographs should be taken from different locations and angles by a moving platform like UAVs, or by a person. It is not enough stereo pair images, it needs to many overlapped images to produce feature model and reconstruct scene geometry (Westoby et al, 2012). During the determination of key points on the overlapped images for image correspondence, Scale Invariant Feature Transform (SIFT) method by Lowe is widely used (1999). By assistance of SIFT algorithm, sensor location and orientation geometry are re-modelled by detection of matching features on the overlapping images. Following detection of key points, camera positions are estimated

by the sparse bundle adjustment system and also sparse point cloud is produced. After minimizing the error of camera positions by nonlinear least square solution; finally, triangulation is performed to reconstruct scene geometry and compute point position. Main advantage of SfM approach over the classic photogrammetry is the full automation of the evaluation process from key point detection to 3D modelling (Westoby et al, 2012). In order to georeferencing, the produced 3D model and manual identification of GCPs are used.

3.4 Aerial Triangulations

Without assistance of GNSS-IMU sensors, air triangulation requires at least 3 GCPs on stereo pair images. By air triangulation it is possible to relate areas where do not contain GCPs (Kraus, 1992). Hence, it is widely preferred by users in terms of economic feasibility. During the process of aerial triangulation all of the orientation parameters are estimated. Tie points are detected to connect each image and blocks in triangulation process. According to Kraus (1992), 6 full GCPs is and 3 height control points are sufficient for estimating orientation parameters of 18 images on the 3 strip. GCPs should be well distributed on the strips, especially they should be locating corners of strips. Recently, block triangulation is commonly performed by using bundle block adjustment (Mikhail et al, 2001).

3.4.1 Bundle block adjustment

It is required a strip of images with 60% overlap and 20% sidelap to perform bundle block adjustment. Bundle of ray represent projection center of photograph and same image coordinates.

Figure 3.6 shows the principle of bundle block adjustment. The image coordinates of tie points which can be seen at more than one photograph and ground coordinates and image coordinates of GCPs are used as initial data. Exterior orientation parameters of each photographs on strip are estimated simultaneously

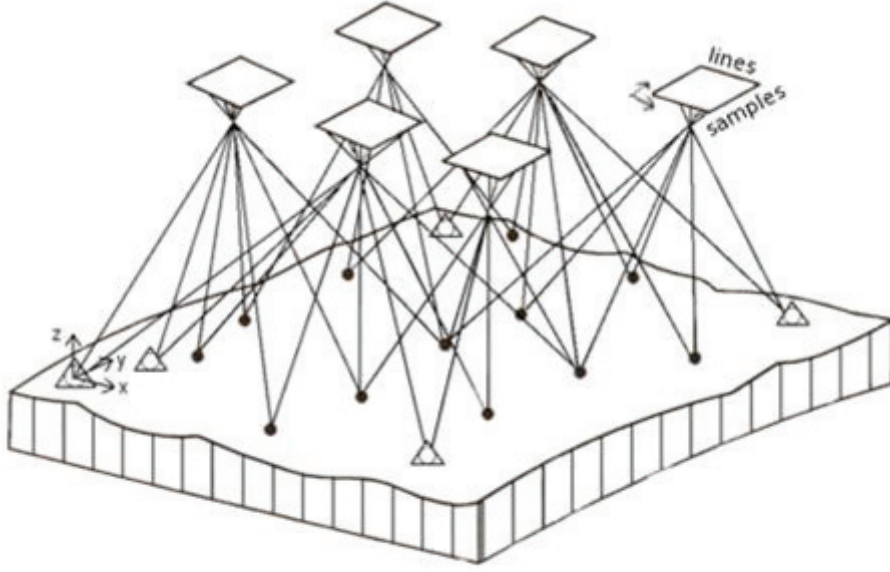


Figure 3.6 : Principal of bundle block adjustment

Collinearity equations; (3.8) and (3.9), are main part of bundle block adjustment. In order to utilize them in the least square adjustment they should be linearized (Mikhail et al, 2001);

$$v + \dot{B}\dot{\delta} + \ddot{B}\ddot{\delta} = f \quad (3.25)$$

where, f is discrepancy vector, v represents residuals, $\dot{\delta}$ contains six parameters of exterior orientation and $\ddot{\delta}$ has three parameters of interior orientation as unknown. $\dot{B}$ and $\ddot{B}$ are known as matrix of partial derivatives of collinearity equations (Mikhail et al, 2001). This correction equation should be written for each point. Commonly least square adjustment is performed to define orientation parameters in terms of forming and solving normal equations.

3.5 Direct Georeferencing

Basically, direct georeferencing (DG) is described as the process of detection of exterior orientation parameters of each images without requirement of conventional aerial triangulation method in context of aerial photogrammetry (Hutton & Mostafa, 2005). Integration of GNSS and IMU sensors on flying vehicle like UAV makes it possible to obtain the ground geodetic coordinates and rotations of aerial images directly. By different satellite positioning techniques likewise; RTK, PPK and PPP the

object coordinates are measured with high accuracy. While obtaining all coordinates and orientations by combined GNSS-IMU system, GNSS positioning measurements are performed to minimize errors in IMU. This combination is referred GPS-aided IMU by American Society for Photogrammetry and Remote Sensing (McGlone, 2004). With direct georeferencing, remote and inaccessible areas can be mapped much easier; it is also reducing costs by means of eliminating requirement of GCPs.

In order to directly obtain object coordinates in geodetic coordinate system, equation (3.26) correlates GNSS-IMU and camera sensor integration terms of location. Initially, it is required to know several systematic parameters on following equation for performing direct georeferencing (Chiang et al, 2015).

$$r_{oA}^g = r_{ou}^g(t) + R_u^g(t) * (s_A * R_i^u * r_{ia}^i + r_{ui}^u) \quad (3.26)$$

where, the r_{oA}^g represents a ground coordinates vector of object point (A), $r_{ou}^g(t)$ is time dependent interpolated ground coordinate vector of GNSS-IMU, s_A refers scale factor, $R_u^g(t)$ is the rotation matrix of GNSS-IMU frame to ground coordinate system, r_{ia}^i is called position vector in i frame, R_i^u is the rotation matrix from image frame to UAV frame, t is time of exposure and lastly r_{ui}^u presents vector between GNSS+IMU and camera projection center (Figure 3.7).

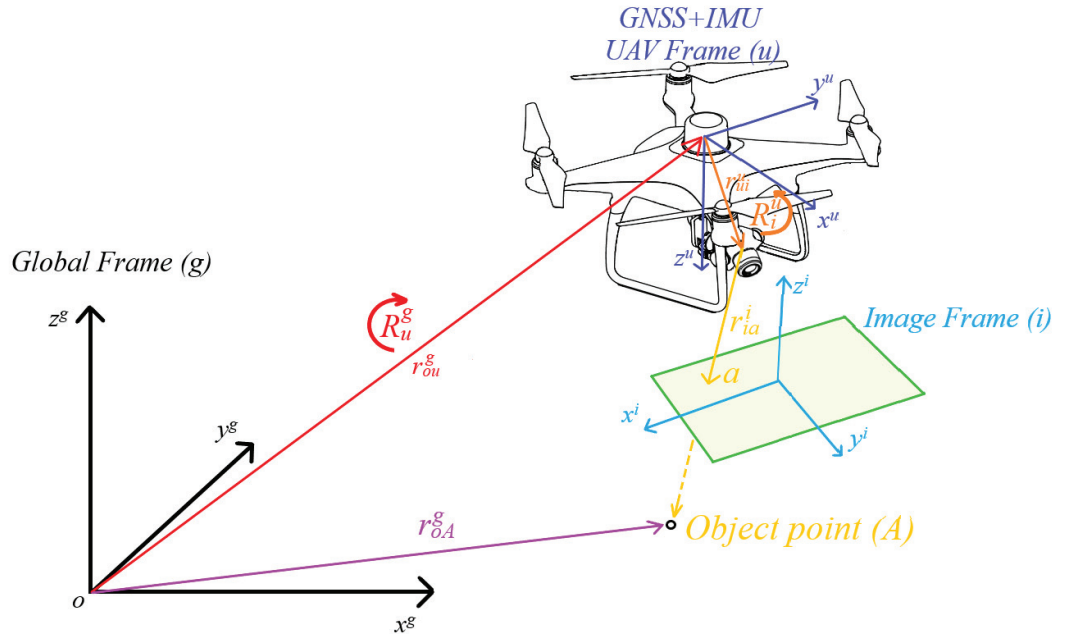


Figure 3.7 : Fundamental relations of direct georeferencing.



4. CASE STUDY

In this study, 948 photographs were used at ITU-Ayazaga Campus. 17 GCPs, which were established by ITU Geomatics Department Photogrammetry Division, measured in situ with static GNSS technique. DJI Phantom 4 RTK were used for flight. During the flight, GNSS receiver located on UAV observed satellites and N-RTK positioned. Acquired photographs were evaluated using automatic evaluation software, Agisoft Metashape Professional. Orthomosaics were produced by global coordinates of camera sensor obtained by N-RTK, PPK and PPP satellite point positioning techniques. Furthermore, aerial triangulation method was applied to create orthophoto using GCPs to compare point accuracy. Hybrid adjustment which contains both exterior orientation and GCPs were performed to clarify the effect of point accuracies on generated models. All outputs were compared with each other. Following Figure 4.1 shows main workflow of UAV based photogrammetry which was followed in this case study.

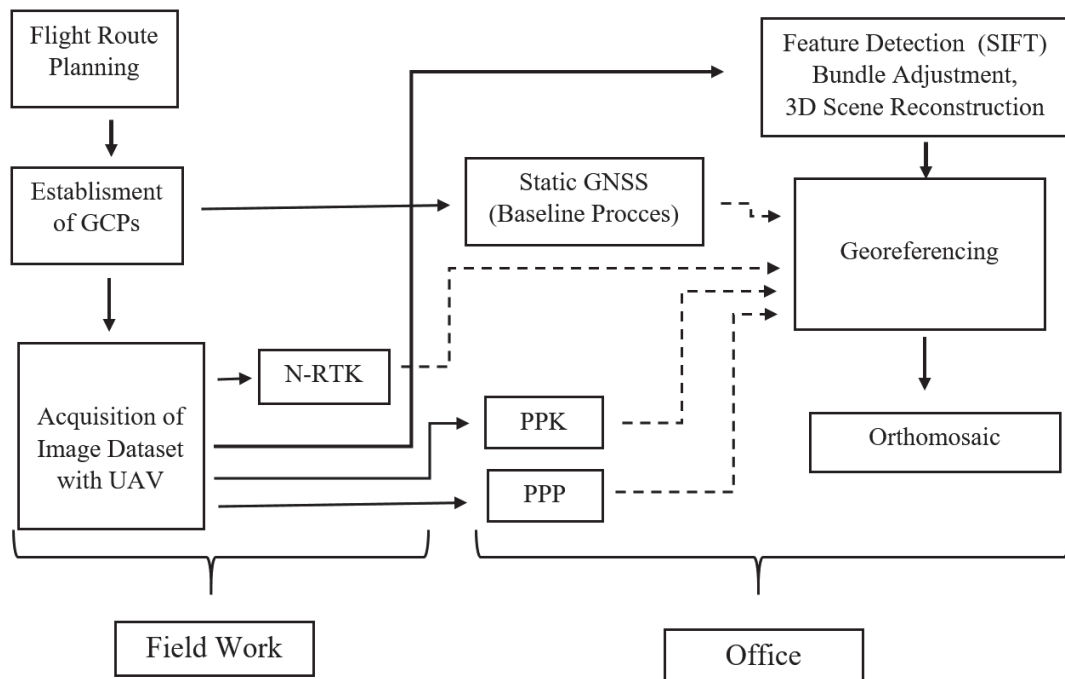


Figure 4.1 : Workflow of UAV Photogrammetry

4.1 Study Area and Establishment of Ground Control Points

Study area is shown in Figure 4.2. It is main campus of Istanbul Technical University called ITU Ayazağa Campus located in Sarıyer, Istanbul. As it is seen, flight is done on the area surrounded by yellow line.

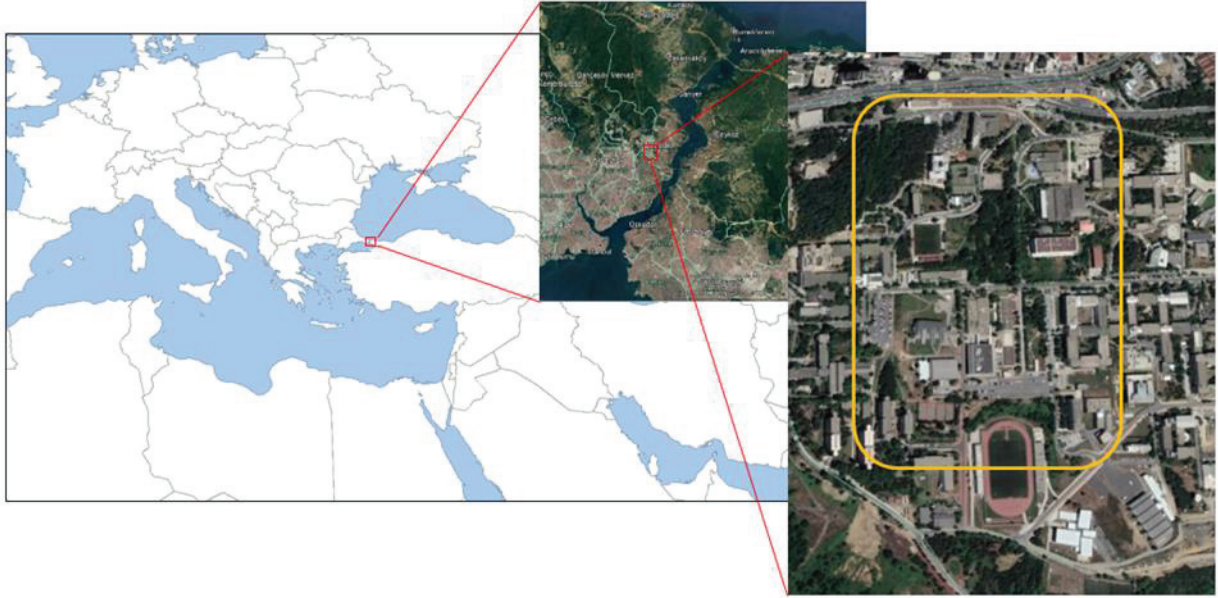


Figure 4.2 : ITU Ayazağa Campus

17 GCPs, which were established by ITU Geomatics Department Photogrammetry Division, measured by relative static GNSS method. Considering the size of the area ($0.45 \text{ km}^2 / 4.5 \text{ ha}$) it was decided to use 17 points. The benchmarks were painted with black and white colour by ITU Geomatics Engineering Department field work students; also, their diameter are in 80 cm (Figure.4.3). Ground Sample Distance (GSD) estimated by the equation (4.1). Centre of GCP is painted with 2 cm in diameter. Flight altitude was about 80 meters from the ground in order to get 2 cm/pixel size.

$$GSD = \frac{\text{Pixel size}}{\text{Focal length}} \times \text{Distance to object} \quad (4.1)$$

GCP coordinates were calculated using Trimble Business Centre commercial software by baseline solution process. During the process, ISKI-UKBS PALA and IGS ISTA points were chosen as reference stations. Baselines between stations and GCP points were solved and points coordinates were obtained by adjustment.



Figure 4.3 : GCPs locations (left) and painted ground control points (right)

Each GCPs coordinates are estimated on ITRF2005 datum in 2005.00 epoch (Table A.1), and also; ITRF 2014 current epoch. RMS values of each points are estimated and average of the horizontal RMS is ± 3 mm and vertical is ± 6 mm (Figure 4.4).

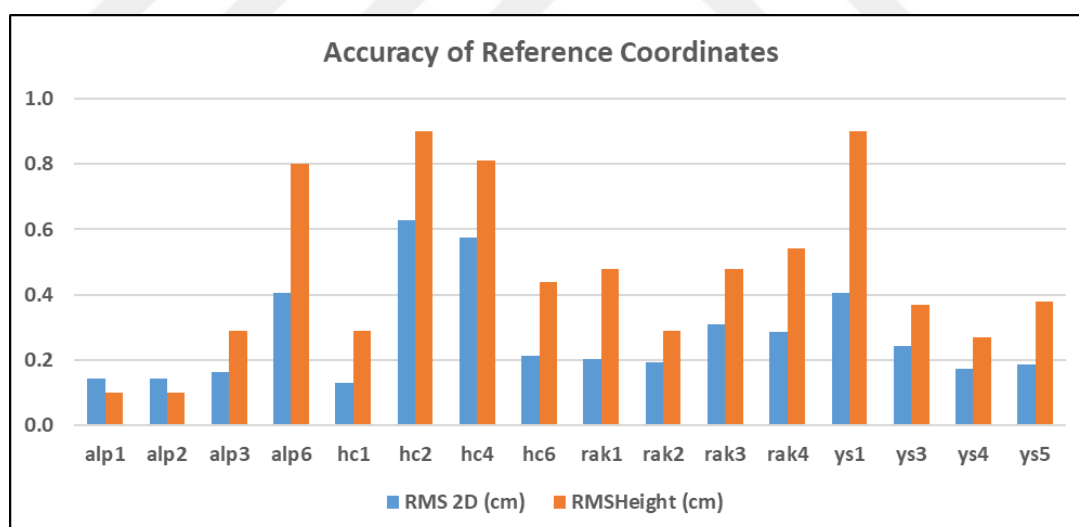


Figure 4.4 : Accuracy of GCPs reference coordinates by Static GNSS relative processing

4.2 UAV Flight and GNSS Measurements

DJI Phantom 4 RTK are multirotor UAV, which were chosen for photogrammetric flight. It has several sensors including magnetic compass, barometer, CMOS”1 camera sensor, IMU which contains gyroscope and accelerometer and dual frequency GNSS receiver (URL3). It is produced by DJI from China. Technical properties are shown in Table 4.1 and, it is seen with control unit in Figure 4.5.

Table 4.1 : Technical Properties of DJI Phantom 4 RTK

Parameter	Properties
Take-off Weight	1391 g
Max Ascent Speed	6 m/s (automatic flight)
Max Flight Time	Approx. 30 minutes
Max Service Ceiling Above Sea Level	19685 ft. (6000 m)
Sensor	1" CMOS; Effective pixels: 20 M
GNSS	GPS : L1/L2 ; GLONASS : L1/L2 ; BeiDou : B1/B2 ; Galileo : E1/E5a
Operating Temperature Range	32° to 104° F (0° to 40°C)



Figure 4.5 : DJI Phantom 4 RTK

During the study, around 948 photographs were taken by UAV in 24 strip (Figure4.6). Flight time was around 2 hours. It was measured in 4 sessions. GNSS receiver was collected data on flight. It has been gathering simultaneously correction for phase and code measurements from RTK network (N-RTK) called ISKI-UKBS CORS network by internet. Using software embed UAV, camera coordinates is measured directly without time interpolation to photograph exposure time and lever arm correction in RTK method.

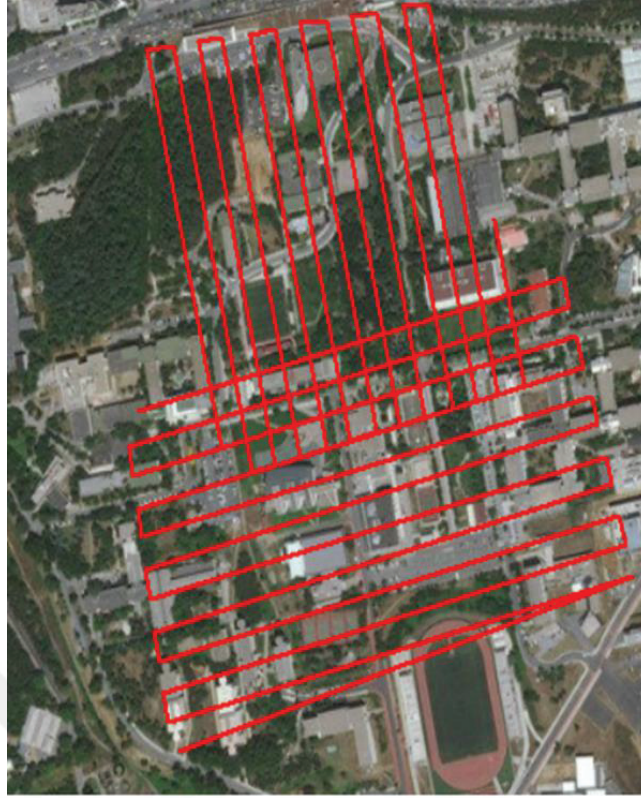


Figure 4.6 : Flight tracks

İSKİ UKBS is real time working a network based on N-RTK method. This network is established in 2008 by water directorate of Istanbul (İSKI) to provide accurate point position determination to users. It covers municipality area of Istanbul with 10 reference station (Figure 4.7). According to examination, it provides ± 2 cm accuracy in horizontal and ± 5 cm accuracy in vertical (URL4).

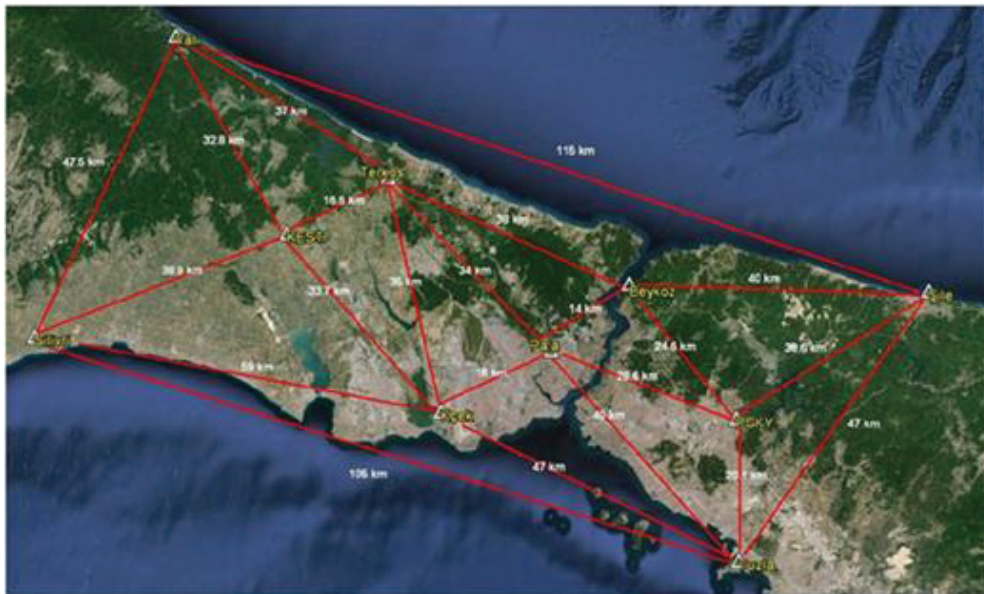


Figure 4.7 : İSKİ UKBS- CORS Network

Beside the N-RTK survey, PPK and PPP methods are applied to determine exterior orientation parameters of each images on the office after field work. As RTK and PPK methods provide relative point positioning, PPP provides absolute point coordinates directly.

Obtained GNSS observation files in RINEX from UAV survey were managed to obtain global geodetic coordinates of each images in PPK process. RINEX data contains 0.2 second sampling interval measurements. These data should be interpolated to coincide exact photograph exposure time. And also, lever arm vector between GNSS antenna and camera sensor should be added on the computation (Equation (3.26)). ISTA IGS station was reference for epoch by epoch baseline solution in the PPK evaluation (Figure 4.8). This application is carried out in CHC Navigation Geomatics Office Software with Drone module.

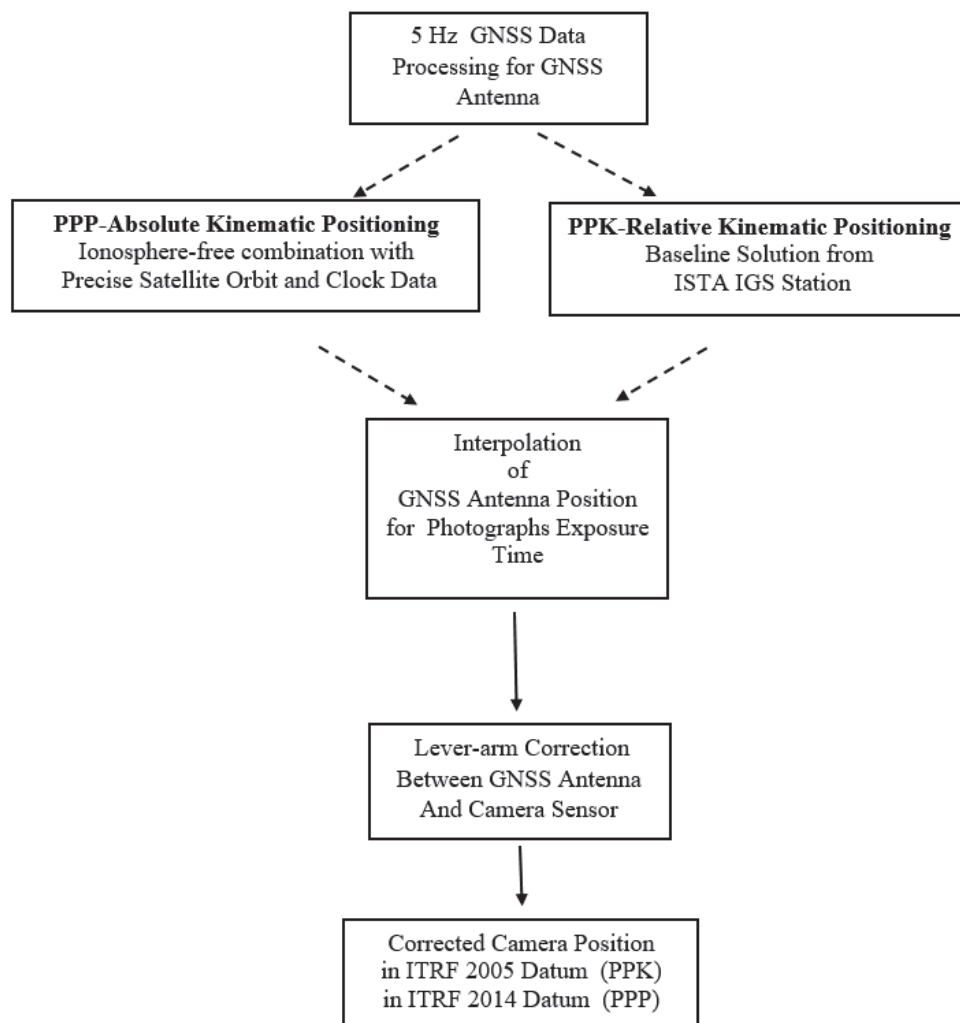


Figure 4.8 : Estimation of camera position with PPK and PPP

On the other hand, PPP method does not provide baseline solution. PPP provides absolute solution to obtain point coordinates. In other word, determining of camera position in geodetic coordinate system does not demand any reference point on PPP method. Around 400 images, which were taken on first flight, were evaluated with PPP technique. As same as with PPK method, solution should be interpolated and lever arm correction should be added GNSS coordinates. PPP process is carried out in online GNSS solution provider called CSRS-PPP (The Canadian Spatial Reference System) (URL5). Interpolated GNSS antenna locations are shown in Figure 4.9.

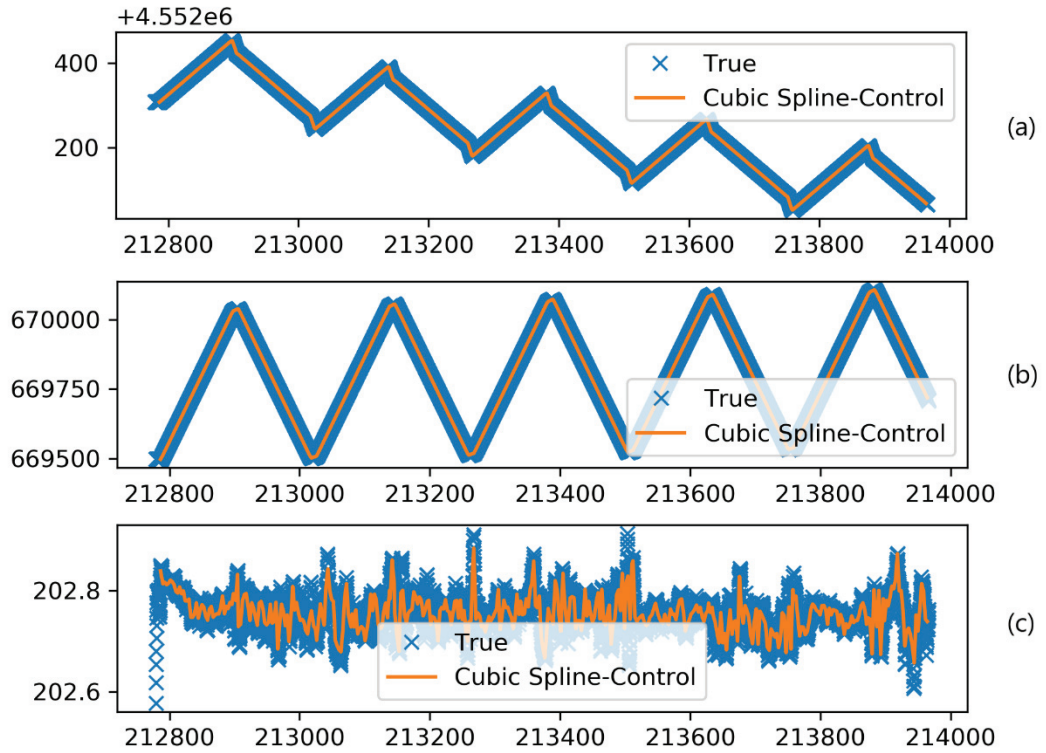


Figure 4.9 : Interpolation of GNSS antenna position (easting (a), northing (b), height (c))

4.3 Photogrammetric Evaluation

Agisoft Metashape Professional software is preferred to evaluate all data. Software is working SfM based algorithm. Firstly, it matches features across the photos by generating a descriptor for each point depending on its local neighborhoods. In terms of this, it uses an algorithm similar to well-known SIFT approach (URL6). Then, to get interior and exterior orientation parameters programme performs bundle block

adjustment. Figure 4.10 represents flow chart of evaluation in AgiSoft Metashape Professional. Image and flight properties are given below Table 4.2.

Table 4.2 : Image and Flight Properties

Number of images	Flying altitude	GSD	Image size	Focal length	Pixel size
948	77.6 m	1.9 cm/pix	5472 x 3648	8.8 mm	2.41 μm

N-RTK, PPK, PPP (camera locations) and Static GNSS (GCPs) points are used to produce orthomosaics. 4 of orthomosaics are evaluated by only individual methods measurements

Models from combined exterior orientation parameters and GCPs coordinates are also produced to see effect of control points on accuracy. Separately, 5 GCPs are merged with RTK, PPK and PPP results for modelling to compare each other. Kraus (1992) stated that, orientation of images by merged data is called hybrid adjustment. In this method, exterior orientations by GNSS-IMU are not used as known coordinates.

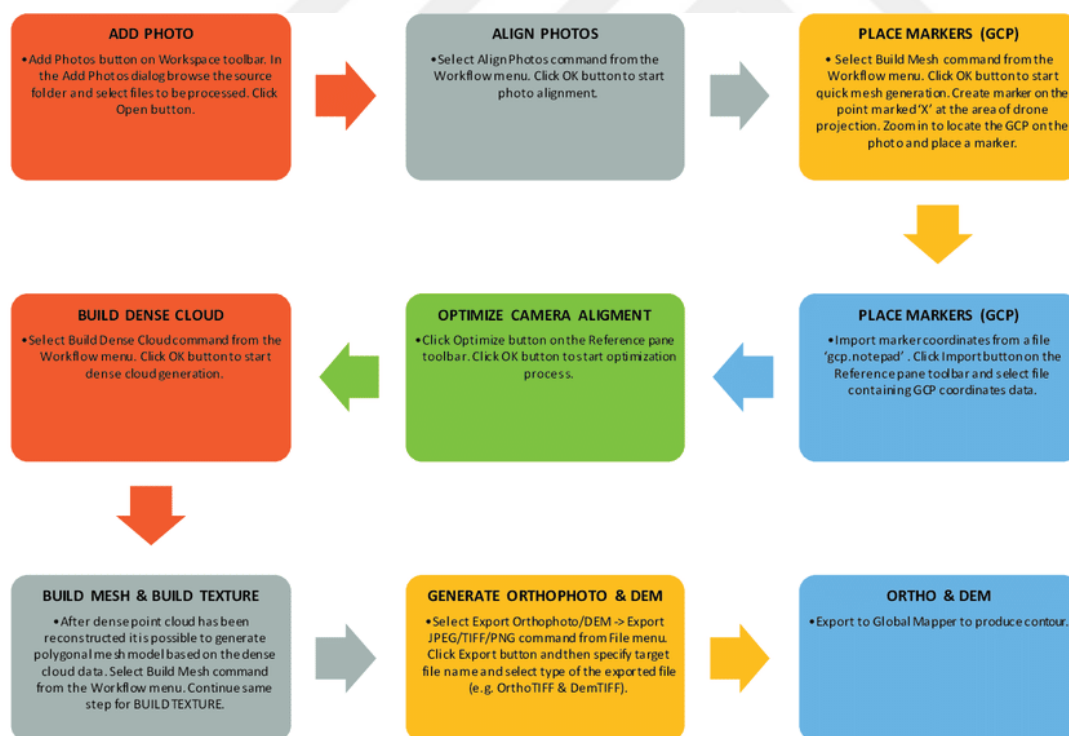


Figure 4.10 : Workflow of Photogrammetric Evaluation (Kumar et al, 2017).

Along the all produced orthomosaics, overlap was %80 and sidelap was %40. Figure 4.11 shows camera locations and overlaps. Totally 7 models are produced by taking measurements of N-RTK, PPK, PPP and 5 GCPs, and also; merged measurements of

N-RTK+5 GCPs, PPK+ 5 GCPs, and PPP+5 GCPs. Table 4.3 represents properties of each produced model.

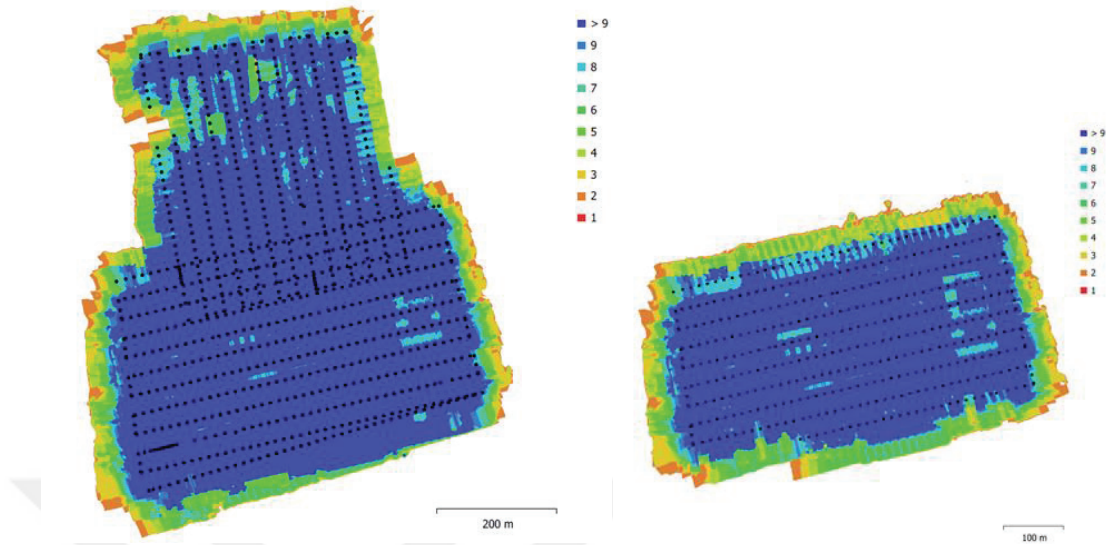


Figure 4.11 : Camera location and image overlaps by colours (produced by 948 and 417 photographs)

Table 4.3 : Properties of each models

Model	Inputs	Number of Images	Aligned Images	Coverage Area	Number of Test Points
N-RTK	Camera Positions	948	941	0.45 km ²	17
PPK	Camera Positions	948	941	0.45 km ²	17
PPP	Camera Positions	417	417	0.26 km ²	13
5 GCPs	Point Positions	948	888	0.42 km ²	12
N-RTK + 5 GCPs	Camera Positions and Point Positions	948	941	0.45 km ²	12
PPK + 5 GCPs	Camera Positions and Point Positions	948	941	0.45 km ²	12
PPP+ 5 GCPs	Camera Positions and Point Positions	417	417	0.26 km ²	8

4.4 Results and Comparisons

GCPs coordinates are obtained from each orthomosaics, which is produced from different methods and their combinations. GCPs coordinates which measured by relative static GNSS method are accepted as absolute coordinates. According to Large Scale Maps and Spatial Data Production Regulation of Turkey, GCPs horizontal accuracy should be lower than $GSD \times 1$ and vertical accuracy should be lower than $GSD \times 1.5$ for aerial photogrammetry (LSMSDPR, 2018). On the other hand, accuracy of points produced by LIDAR technique should be ± 7 cm in horizontal and vertical,

according to regulation. Estimated mean GCPs position accuracies are 3 mm horizontal and 6 mm vertical (Figure 4.4). So, it is clearly seen that relative static GNSS measurements satisfy the criterion for accuracy of GCP by the regulation.

In this section, some figures about N-RTK, PPK, PPP and 5 GCPs models outputs are given Hybrid models figures and tables are added on Appendix A.

There are two photogrammetric models produced from only N-RTK survey data and combination of GCPs; N-RTK+5 GCPs. Figure 4.12 gives the differences between absolute coordinates (relative static GNSS) and GCPs coordinates extracted from orthomosaic produced by N-RTK measurements. Extracted GCPs coordinates, and illustrations of horizontal and vertical differences from absolute coordinates are added on Appendix A.

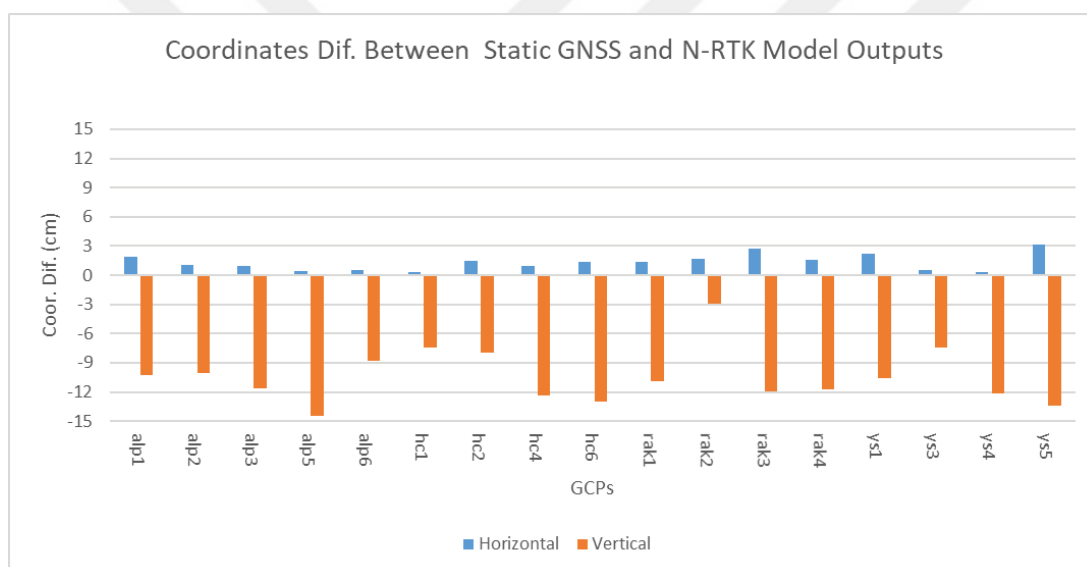


Figure 4.12 : Coordinates differences between static GNSS and N-RTK model outputs

Because of GSD, approximately 2 cm in diameter, position accuracy of painted GCPs extracted from the orthomosaics is expected ± 3 cm in horizontal. Besides, ± 5 cm accuracy is expected in vertical based on service provider (ISKI-UKBS) accuracy with N-RTK method. Obviously, it is seen that there is bias on the N-RTK vertical position. As, all vertical differences are in same direction and their values are close to each other. But then, horizontal differences come in sight as expected.

And also, two different models are produced from PPK positions and combination of GCPs data. Using PPK data, it is expected more accurate results than N-RTK models. Since, N-RTK values are obtained simultaneously while survey in situ. PPK data is

processed in office, so that, it is enable to see and remove biases and blunders. Also, it is thought that distance from reference station (ISTA IGS) is strongly effects accuracy. Figure 4.13 indicates differences between PPK models and absolute coordinates. Coordinates and illustrations of differences are added in Appendix A.

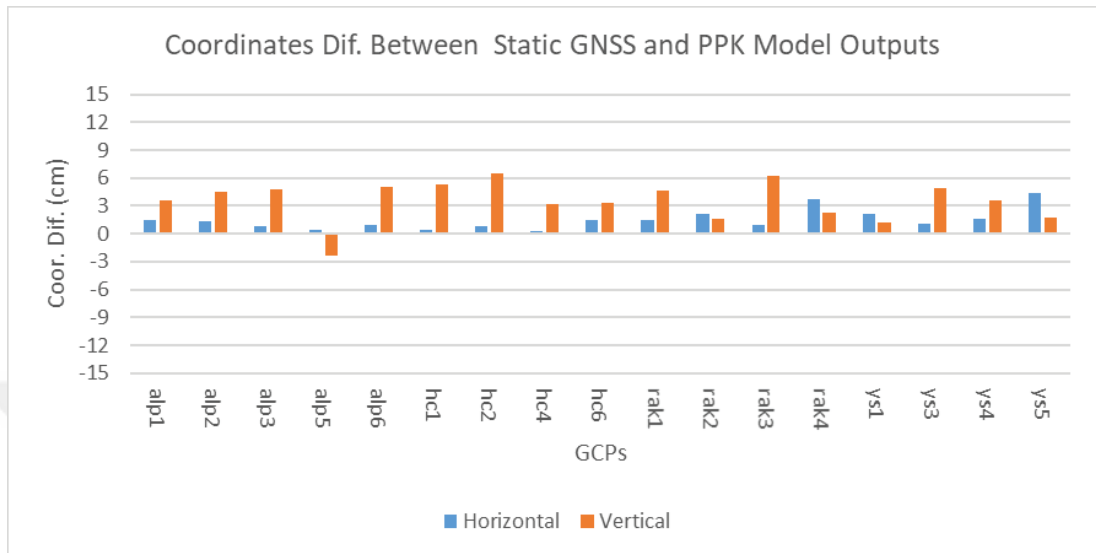


Figure 4.13 : Coordinates differences between static GNSS and PPK model outputs

The last performed satellite based positioning method is PPP method that its results are expected in decimeter level accuracy. In this measurement, it is only evaluated first session flight images. Area which covered by 417 photographs contains 13 GCPs. Two different photogrammetric model is produced by image coordinates obtained by PPP process and, combination of image coordinates by PPP and 5 GCPs. In that case, all coordinates are on the ITRF 2014 datum. Comparison between absolute coordinates and coordinates obtained from PPP model are shown Figure 4.14 given below.

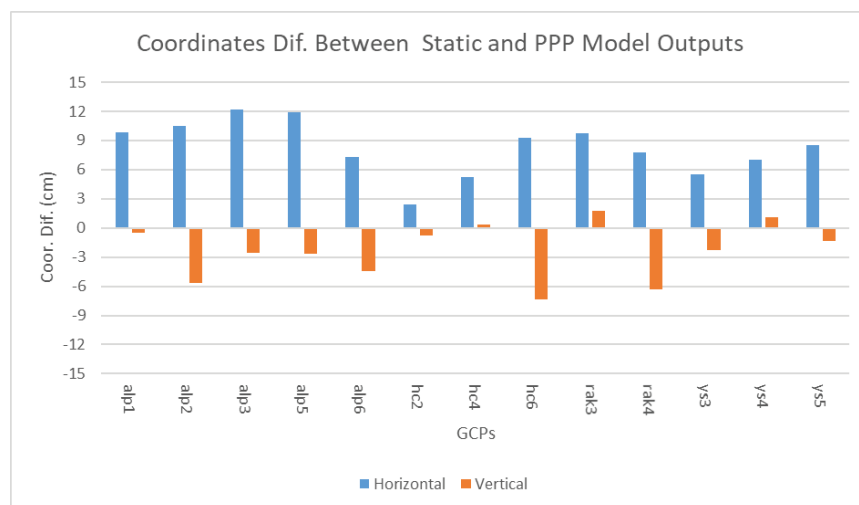


Figure 4.14 : Coordinates differences between static GNSS and PPP model outputs

As it is seen, horizontal differences come into view as prediction. Yet, interestingly vertical differences are smaller than horizontal differences. Because, PPP coordinates are estimated in current datum and epoch, GCPs coordinates are transformed from ITRF 2005 to ITRF 2014, also; epoch velocities are added. It is assumed that high horizontal differences stem from transformation and velocity corrections to current epoch. To compare direct-georeferencing with aerial triangulation, one more models produced by using only 5 GCPs (Figure 4.15).

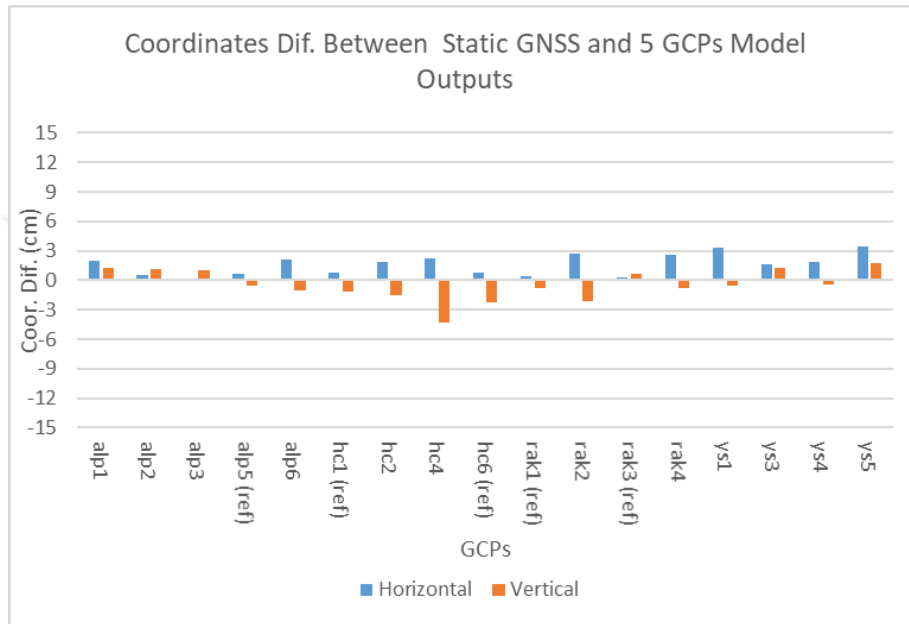


Figure 4.15 : Coordinates differences. between static GNSS and 5 GCPs Model Outputs

When, absolute coordinates were compared with coordinates extracted from models produced by aerial triangulation, it is seen that results were consistent with estimations. Horizontal and vertical differences are under the 3 cm.

All models statistics computed between model coordinates and absolute static coordinates of GCPs are shown in Table 4.4. Also, Figure 4.15 illustrates mean, standard deviation (STD) and root mean square (RMS) errors of both horizontal and vertical coordinate differences. All model comparisons, extracted GCPs coordinates and horizontal and vertical differences illustrations are given on Appendix A.

If models which evaluated by N-RTK measurements and its combination are compared between each other, it is clearly seen that vertical accuracies are better in the hybrid model combined with GCPs.

While comparing PPK models in horizontal, GCPs combinations affected results negatively. Considering accuracy of test points horizontal coordinates (northing, easting) from orthomosaics are 3 cm because of GSD (2 cm). That's why there are no huge differences between them. Also, results of vertical coordinate differences are better as predicted. Moreover, it is assumed that. there is outlier between PPK and relative static measurement.

In other side, PPP results are very promising except horizontal bias because of transformation and epoch correction. Vertical differences are yet much better than expected. Besides, results of its combination with GCPs are much better than results of other models.

Over all, if all models from different techniques; RTK, PPK and PPP, are compared, best results are obtained from PPP models. However, it is considered area which PPP performed better; also, geometry of GCPs in this area are better than whole area. Hence, it is thought PPP outputs are more reliable. Though, PPK outputs are quite usable for applications. If biases on the RTK measurements are removed, RTK can be used for applications as well.

Table 4.4 : Statistics of Horizontal and Vertical Differences

Model	Number of Test Points	Statistics of Differences Between Model Outputs and Static GNSS Measurement (cm)					
			Min	Max	Mean	STD	RMS
RTK	17	Horizontal	0.3	3.1	1.3	0.8	1.6
		Vertical	-14.5	-2.9	-10.4	2.8	10.8
PPK	17	Horizontal	0.2	4.4	1.5	1.1	1.9
		Vertical	-2.3	6.5	3.5	2.2	4.2
PPP	13	Horizontal	2.4	12.2	8.2	2.8	8.7
		Vertical	-7.4	1.7	-2.4	2.9	3.7
RTK + 5 GCPs	12	Horizontal	0.3	3.0	1.4	0.9	1.6
		Vertical	-3.9	5.9	-0.4	2.7	2.7
PPK + 5 GCPs	12	Horizontal	1.1	4.0	2.5	1.1	2.7
		Vertical	-7.3	6.4	-0.8	3.8	3.8
PPP + 5 GCPs	8	Horizontal	0.2	1.4	0.8	0.5	0.9
		Vertical	-3.0	3.0	-0.5	1.8	1.9
5 GCPs	12	Horizontal	0.2	3.5	2.0	1.0	2.3
		Vertical	-4.3	1.7	-0.4	1.7	1.7

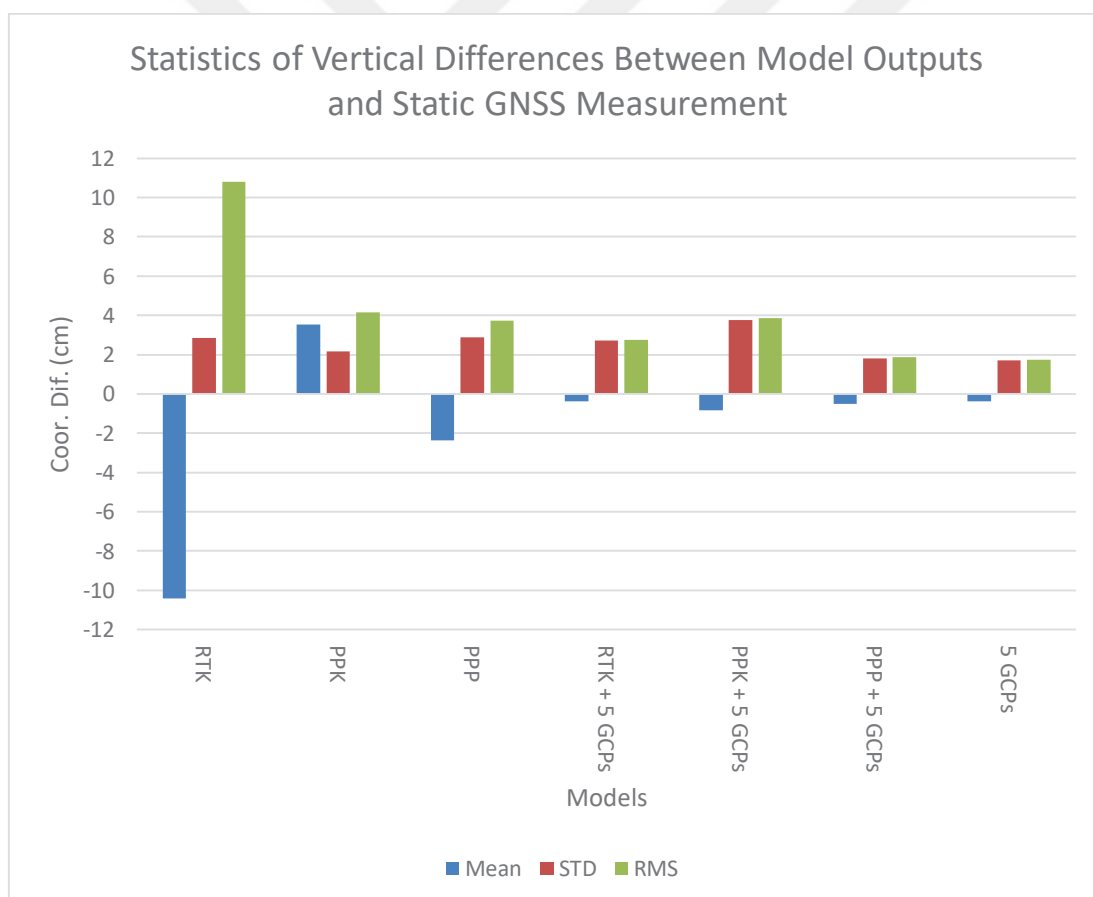
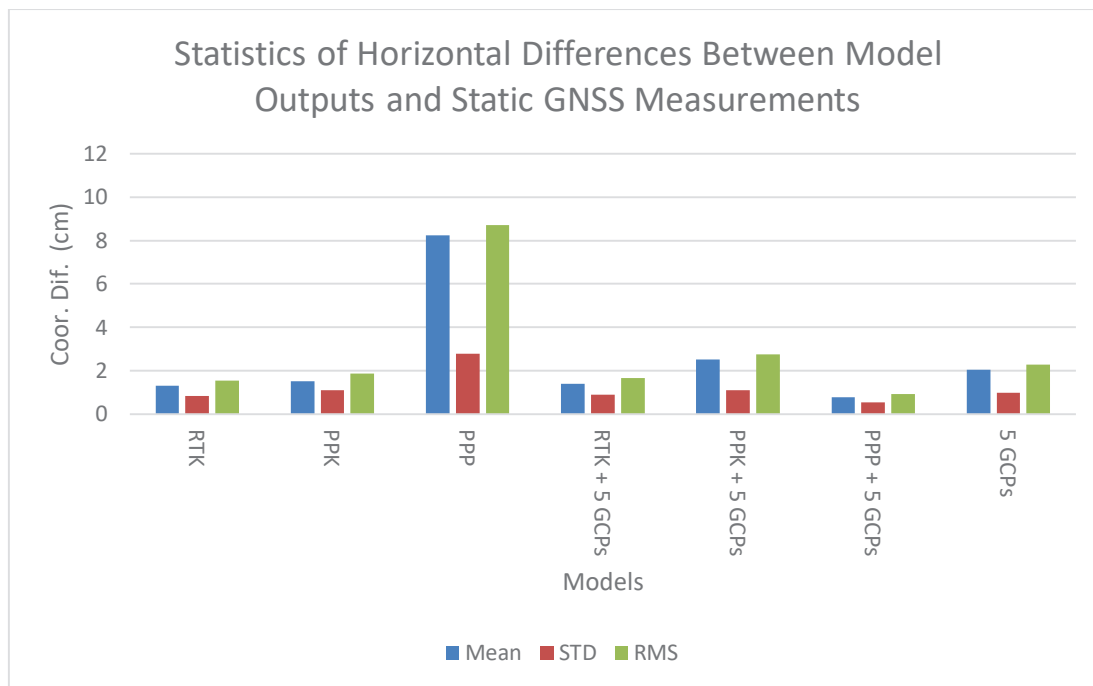


Figure 4.16 : Statistics of horizontal (above) and vertical (below) differences

5. CONCLUSION AND RECOMMENDATIONS

In this study; it was aimed to assess the effects of point positioning accuracy on accuracy in photogrammetric productions using UAVs. With this purpose, N-RTK, PPK, PPP methods were used to produce orthomosaics applied for calculating the point coordinates and hence the role of the obtained accuracies of the reference and test points on the produced photogrammetric models were evaluated. By performing different approaches, it is aimed to eliminate dependency of GCP's and evaluate performance of satellite positioning techniques on the photogrammetric mapping. Elimination of GCP dependency is essential factor to reach disaster areas; also, hazardous mine and industrial areas. Within this scope, 17 ground points were measured in ITU-Ayazağa Campus, size of study area is 455,000 m². Points were set up to use as a control and test points. Every points were measured by relative static GNSS methods to obtain under 1 cm accuracy. As a result of evaluation, the mean accuracy of horizontal position is found out around the 3 mm and vertical position mean accuracy around the 6 mm.

In context of study, if it is compared only 2 model by N-RTK outputs, best option is N-RTK + 5 GCPs model (Figure 4.15). By GCPs, vertical and horizontal positions are improved. Though, it is thought that there are some errors arisen from relative static GNSS measurements especially at points rak2 and alp5. If biases on the height measurements which are belong to N-RTK model can be removed, it can be possible to obtain better results. And also, it is thought that errors arisen from RTK network and network accuracy reduce measurements accuracy.

Both PPP and PPK methods gave similar results by means of horizontal and vertical component comparisons. Although, there are errors originated from datum transformation and epoch corrections, high accuracy can be obtained with PPP by means of GCPs. In fact, it was not expected such a good result by PPP, while comparing with each other. It can be derived from small area evaluation and well distributed GCPs.

If standard deviation of all models are compared, all three satellite positioning techniques results are close enough to criterion described in regulation. According to map production regulation in Turkey test point accuracies should be smaller than GSD x 1.5 in horizontal and GSD x 2 in vertical (LSMSDPR, 2018). Yet, if biases in RTK and PPP measurements can be removed, they can give comparable results in case of root mean square errors.

Another important argument is relative static GNSS measurement and estimation of GCPs. Although, inner accuracy of the measurement has higher accuracy, it is assumed that it has outlier with N-RTK and PPK outputs.

To summarize, compared to full size manned aerial vehicles, UAV based photogrammetry has advantages for special tasks in many aspects like; time, money, small size (portability), rapid and accurate data acquisition, and reaching remote and danger areas without pilot on-board. Furthermore, it provides higher spatial resolution with integration of sensors and data acquisition. However, it has some limitations as well, such as; limited payload and battery. Using the observations by GNSS-IMU, the necessity of establishing GCP can be overcome by using RTK, PPK and PPP methods in order to provide necessary accuracy. Providing direct exterior orientation parameters via RTK methods reduces office works; nevertheless, data qualities can be increased in office by PPK and PPP methods. It is still depending on user priorities. On the other hand, it is clearly seen that GCPs improves model qualities. It is recommended that using at least one GCP on UAV based photogrammetry to eliminate noises and biases to get higher accuracy in vertical. For future studies, it is advised to perform single RTK methods on the area with well distributed GCPs without biases in order to see effects of N-RTK errors on the measurements. In addition, differences in elevation was not too high in study area. UAV based photogrammetry may be applied on areas such as mountain and mining areas where topography changes are dramatically high. It can be investigating vertical accuracies using different GNSS techniques.

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APPENDICES

APPENDIX A: Measured and extracted coordinates of GCPs from orthophotos and illustrations of coordinate differences



APPENDIX A

Table A.1 : GCPs coordinates from relative static GNSS measurement

Points	Northing (m)	Easting(m)	H(m)
alp1	4552579.201	417712.8617	131.6964
alp2	4552593.199	417745.5474	131.6717
alp3	4552520.873	417693.9343	127.7662
alp5	4552619.884	417681.9674	129.9377
alp6	4552636.158	417793.9427	132.2198
hc1	4552786.568	417760.933	143.9057
hc2	4552600.963	417981.3623	130.033
hc4	4552521.545	418011.6959	128.8803
hc6	4552510.917	417825.2006	130.6044
rak1	4552978.747	417862.0268	150.845
rak2	4552894.596	417985.3464	153.8048
rak3	4552461.321	418010.9297	127.9104
rak4	4552510.205	417784.4451	130.4034
ys1	4552909.762	417805.5745	147.7873
ys3	4552538.554	417941.9548	129.0475
ys4	4552471.419	417951.5637	127.9667
ys5	4552403.303	417855.3018	124.3721

Table A.2 : GCPs coordinates from models produced by N-RTK

Points	Easting(m)	Northing(m)	H(m)
alp1	417712.866	4552579.182	131.799
alp2	417745.550	4552593.189	131.773
alp3	417693.936	4552520.864	127.883
alp5	417681.967	4552619.880	130.083
alp6	417793.938	4552636.155	132.308
hc1	417760.936	4552786.566	143.980
hc2	417981.353	4552600.975	130.113
hc4	418011.690	4552521.553	129.004
hc6	417825.214	4552510.921	130.735
rak1	417862.040	4552978.745	150.954
rak2	417985.342	4552894.612	153.834
rak3	418010.935	4552461.348	128.030
rak4	417784.442	4552510.189	130.521
ys1	417805.555	4552909.772	147.893
ys3	417941.959	4552538.556	129.122
ys4	417951.560	4552471.420	128.088
ys5	417855.301	4552403.272	124.506

Table A.3 : GCPs coordinates from models produced by N-RTK + 5 GCPs

Points	Easting(m)	Northing(m)	H(m)
alp1	417712.866	4552579.183	131.702
alp2	417745.556	4552593.198	131.671
alp3	417693.935	4552520.865	127.800
alp5	417681.968	4552619.879	130.007
alp6	417793.939	4552636.153	132.237
hc1	417760.933	4552786.565	143.915
hc2	417981.356	4552600.974	130.031
hc4	418011.690	4552521.556	128.896
hc6	417825.211	4552510.919	130.641
rak1	417862.044	4552978.744	150.836
rak2	417985.343	4552894.607	153.746
rak3	418010.940	4552461.344	127.902
rak4	417784.436	4552510.186	130.421
ys1	417805.546	4552909.771	147.793
ys3	417941.958	4552538.549	129.006
ys4	417951.567	4552471.418	127.979
ys5	417855.305	4552403.275	124.412

Table A.4 : GCPs coordinates from models produced by PPK

Points	Easting(m)	Northing(m)	H(m)
alp1	417712.870	4552579.189	131.660
alp2	417745.559	4552593.205	131.626
alp3	417693.937	4552520.865	127.718
alp5	417681.963	4552619.884	129.961
alp6	417793.937	4552636.166	132.169
hc1	417760.933	4552786.573	143.853
hc2	417981.357	4552600.969	129.968
hc4	418011.694	4552521.544	128.849
hc6	417825.215	4552510.912	130.572
rak1	417862.033	4552978.733	150.798
rak2	417985.327	4552894.602	153.788
rak3	418010.939	4552461.323	127.848
rak4	417784.432	4552510.170	130.381
ys1	417805.554	4552909.767	147.775
ys3	417941.963	4552538.546	128.998
ys4	417951.560	4552471.404	127.930
ys5	417855.304	4552403.259	124.354

Table A.5 : GCPs coordinates from models produced by PPK + 5 GCPs

Points	Easting(m)	Northing(m)	H(m)
alp1	417712.853	4552579.174	131.672
alp2	417745.544	4552593.188	131.670
alp3	417693.928	4552520.863	127.762
alp5	417681.964	4552619.872	129.982
alp6	417793.925	4552636.152	132.244
hc1	417760.927	4552786.561	143.891
hc2	417981.347	4552600.951	130.094
hc4	418011.668	4552521.542	128.928
hc6	417825.197	4552510.913	130.616
rak1	417862.039	4552978.751	150.862
rak2	417985.309	4552894.612	153.878
rak3	418010.924	4552461.325	127.901
rak4	417784.424	4552510.181	130.381
ys1	417805.540	4552909.779	147.804
ys3	417941.946	4552538.547	129.032
ys4	417951.544	4552471.407	127.976
ys5	417855.277	4552403.273	124.308

Table A.6 : GCPs coordinates from models produced by PPP

Points	Easting(m)	Northing(m)	H(m)
alp1	417713.333	4552579.352	131.720
alp2	417746.025	4552593.364	131.747
alp3	417694.426	4552521.048	127.810
alp5	417682.456	4552620.009	129.983
alp6	417794.387	4552636.299	132.283
hc2	417981.742	4552601.137	130.059
hc4	418012.087	4552521.744	128.895
hc6	417825.658	4552511.105	130.697
rak3	418011.325	4552461.567	127.911
rak4	417784.894	4552510.370	130.485
ys3	417942.372	4552538.738	129.089
ys4	417951.974	4552471.630	127.974
ys5	417855.746	4552403.500	124.404

Table A.7 : GCPs coordinates from models produced by PPP + 5 GCPs

Points	Easting(m)	Northing(m)	H(m)
alp1	417713.235	4552579.349	131.710
alp2	417745.923	4552593.350	131.681
alp3	417694.312	4552521.024	127.797
alp5	417682.349	4552620.040	129.926
alp6	417794.316	4552636.318	132.208
hc2	417981.735	4552601.108	130.074
hc4	418012.059	4552521.686	128.916
hc6	417825.581	4552511.072	130.653
rak3	418011.299	4552461.480	127.955
rak4	417784.805	4552510.343	130.417
ys3	417942.327	4552538.691	129.076
ys4	417951.926	4552471.562	128.005
ys5	417855.658	4552403.437	124.403

Table A.8 : GCPs coordinates from models produced by 5 GCPs

Points	Easting(m)	Northing(m)	H(m)
alp1	417712.855	4552579.182	131.683
alp2	417745.546	4552593.194	131.661
alp3	417693.934	4552520.871	127.756
alp5	417681.970	4552619.878	129.944
alp6	417793.927	4552636.145	132.230
hc1	417760.932	4552786.560	143.917
hc2	417981.348	4552600.950	130.048
hc4	418011.676	4552521.534	128.924
hc6	417825.199	4552510.909	130.627
rak1	417862.030	4552978.745	150.853
rak2	417985.326	4552894.614	153.826
rak3	418010.932	4552461.321	127.904
rak4	417784.429	4552510.185	130.411
ys1	417805.545	4552909.776	147.793
ys3	417941.951	4552538.539	129.035
ys4	417951.551	4552471.405	127.971
ys5	417855.276	4552403.280	124.355



Figure A.1 : Horizontal differences between static GNSS and N-RTK model outputs

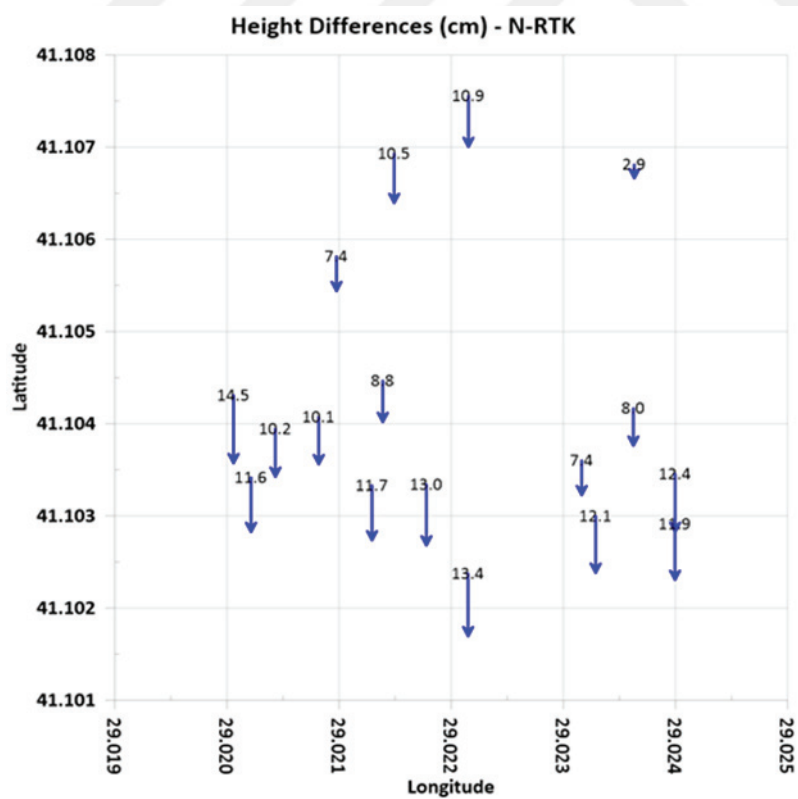


Figure A.2 : Vertical differences between static GNSS and N-RTK model outputs

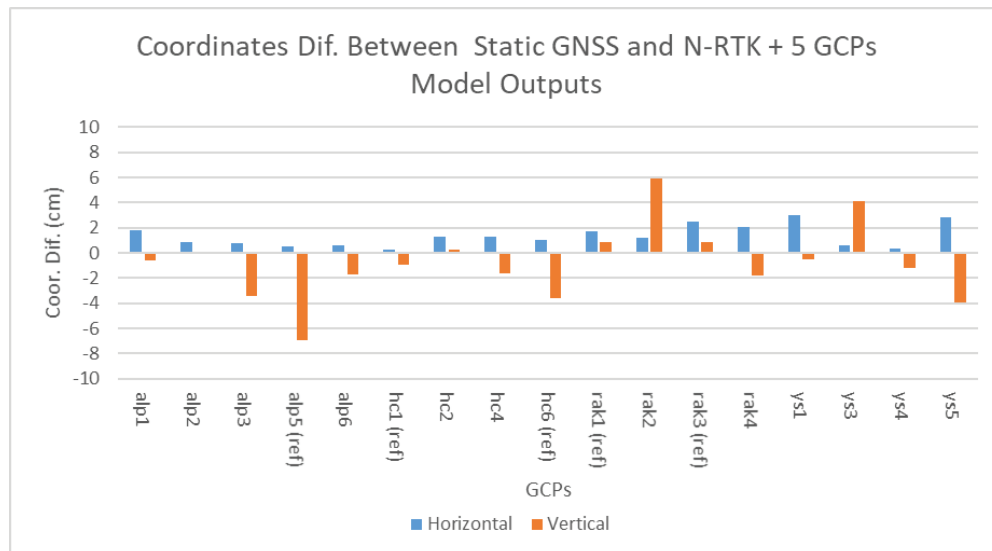


Figure A.3 : Coordinates differences between static GNSS and N-RTK + 5 GCPs model outputs

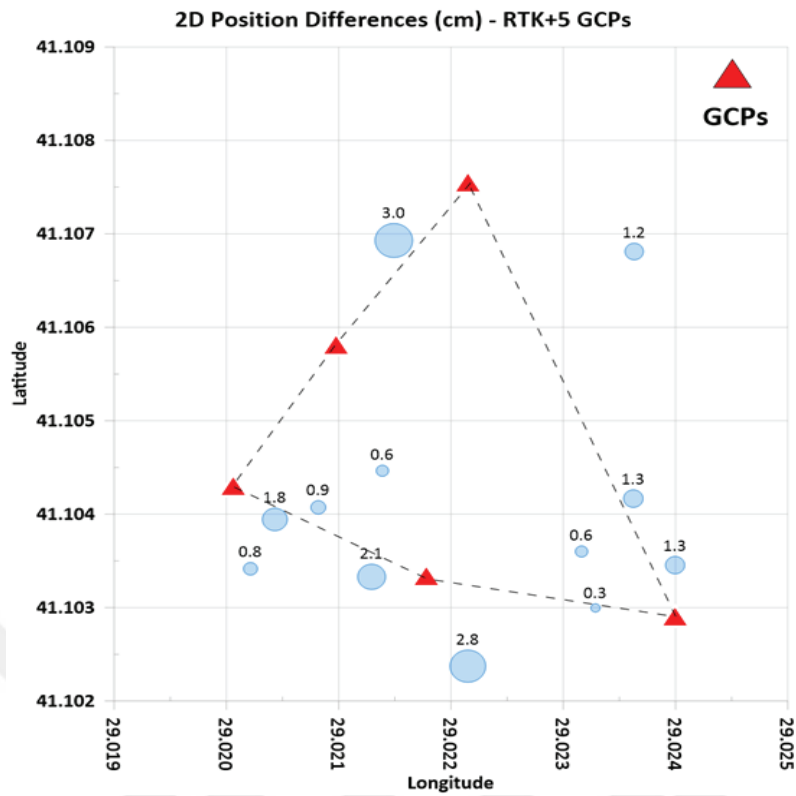


Figure A.4 : Horizontal differences between static GNSS and N-RTK + 5 GCPs model outputs

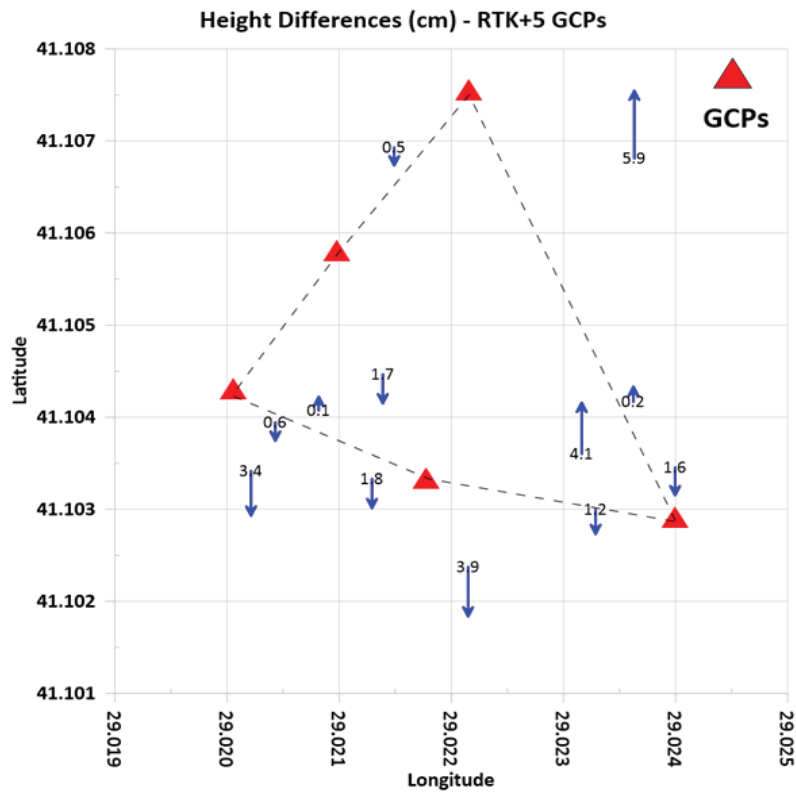


Figure A.5 : Vertical differences between static GNSS and N-RTK + 5 GCPs model outputs

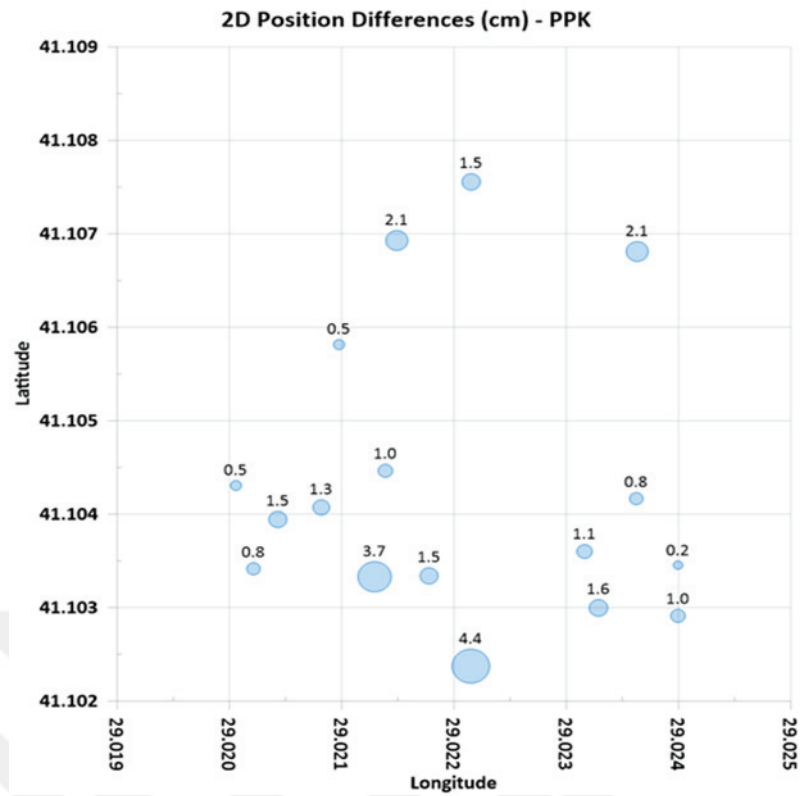


Figure A.6 : Horizontal differences between static GNSS and PPK model outputs

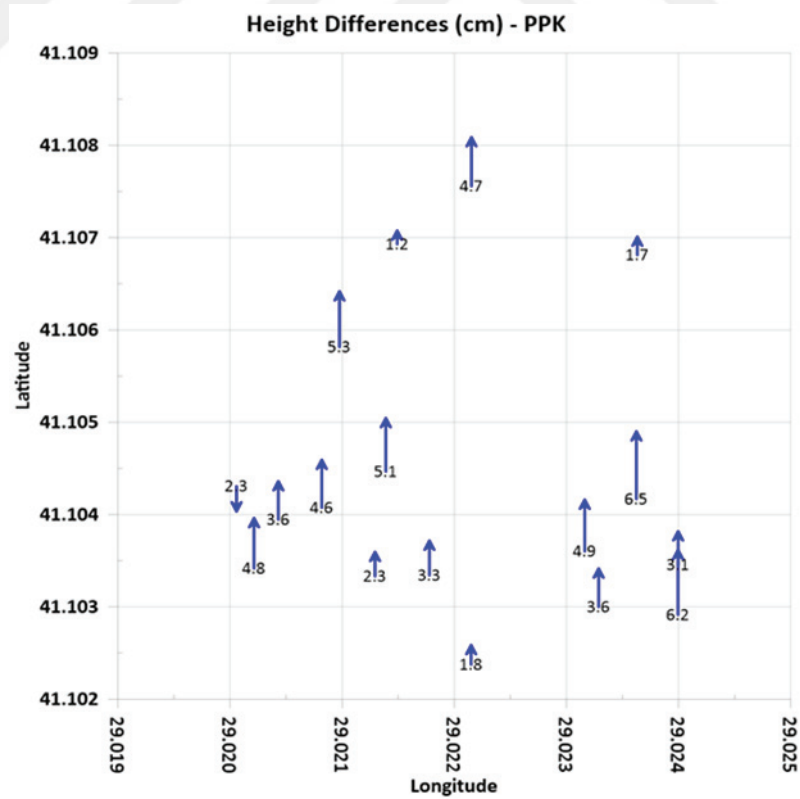


Figure A.7 : Vertical differences between static GNSS and PPK model outputs

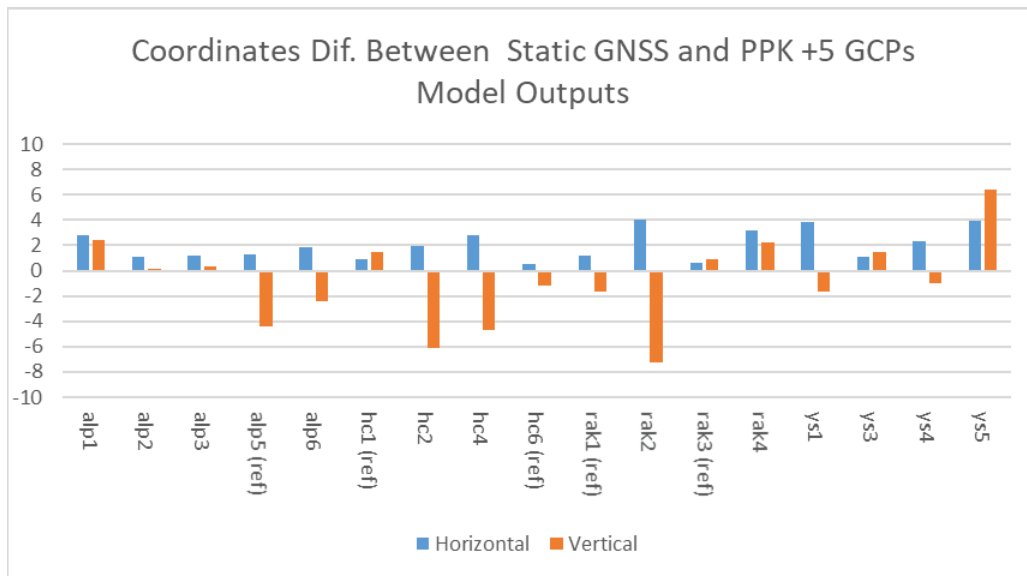


Figure A.8 : Coordinates differences between static GNSS and PPK + 5 GCPs model outputs

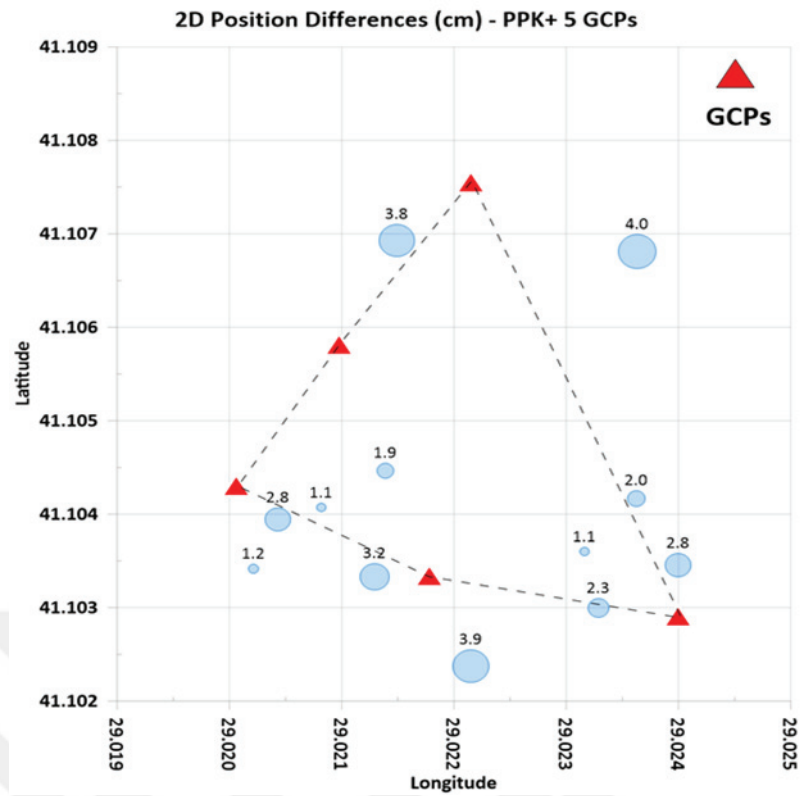


Figure A.9 : Horizontal differences between static GNSS and PPK + 5 GCPs Model outputs

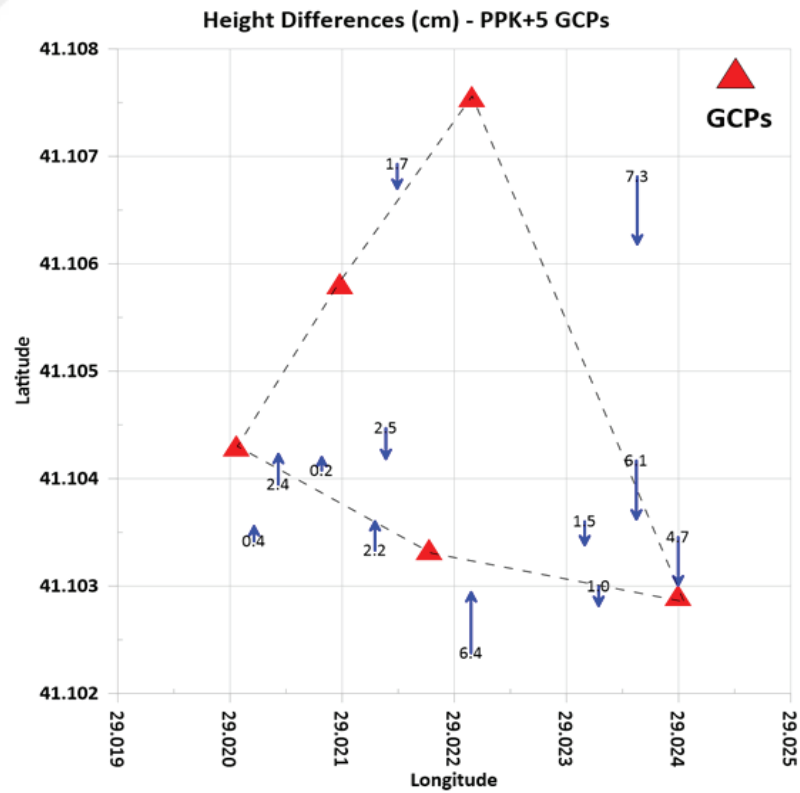


Figure A.10 : Vertical differences between static GNSS and PPK + 5 GCPs model outputs

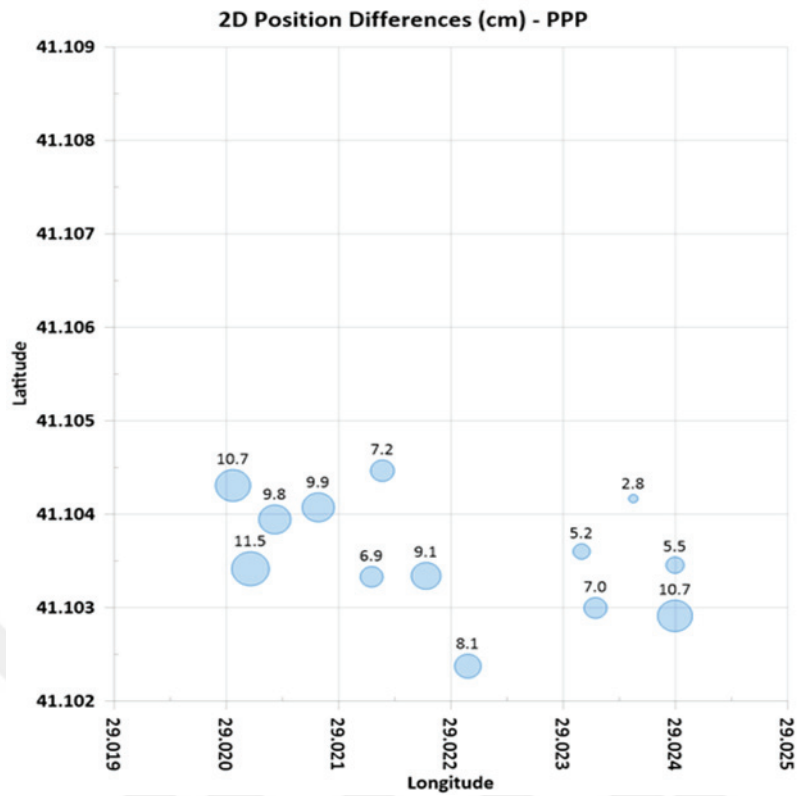


Figure A.11 : Horizontal differences between static GNSS and PPP model outputs

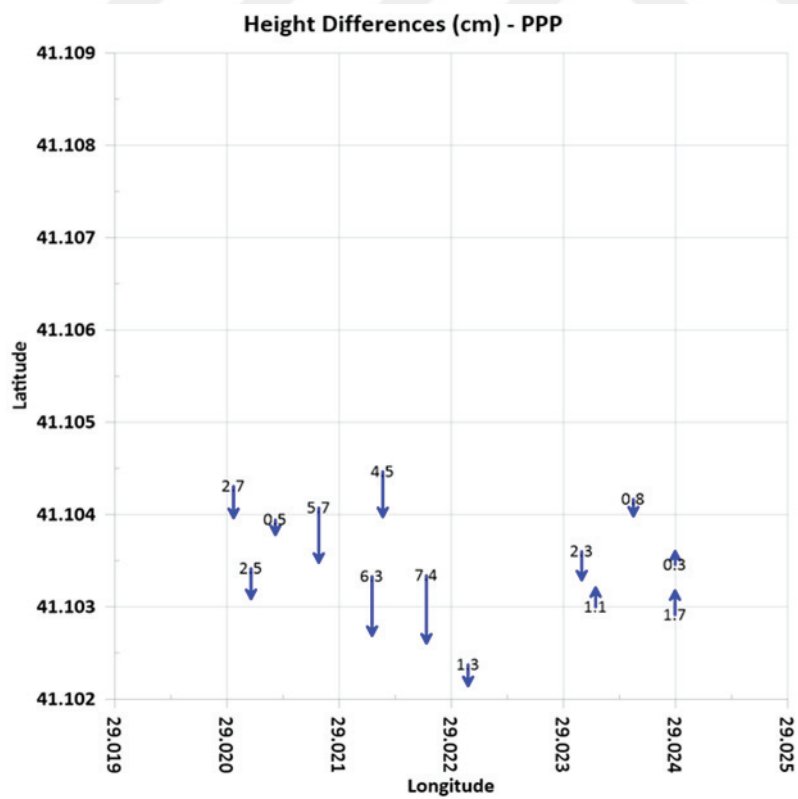


Figure A.12 : Vertical differences between static GNSS and PPP model outputs

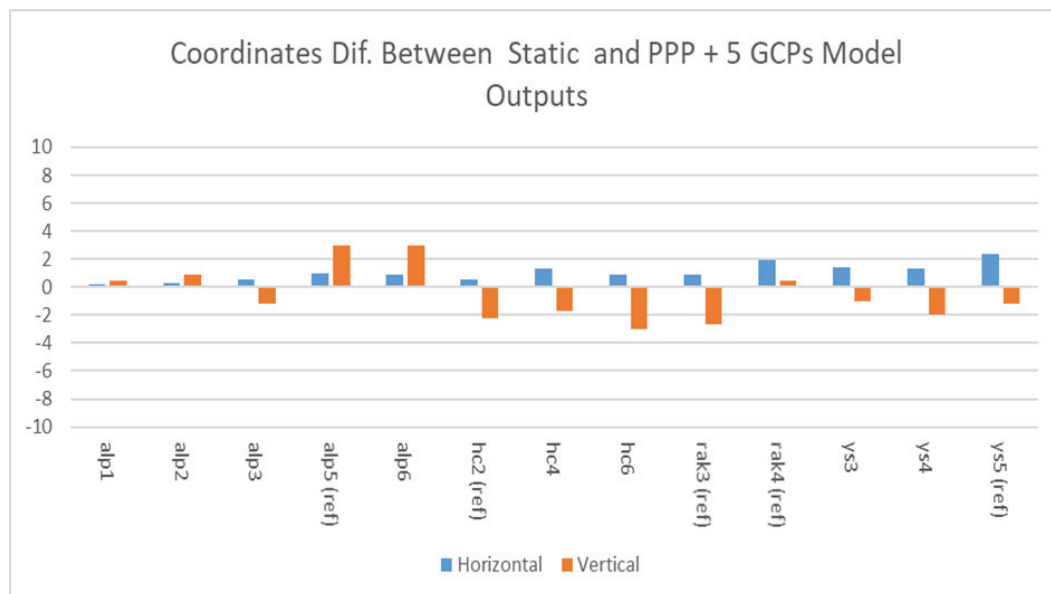


Figure A.13 : Coordinates differences between static GNSS and PPP + 5 GCPs model outputs

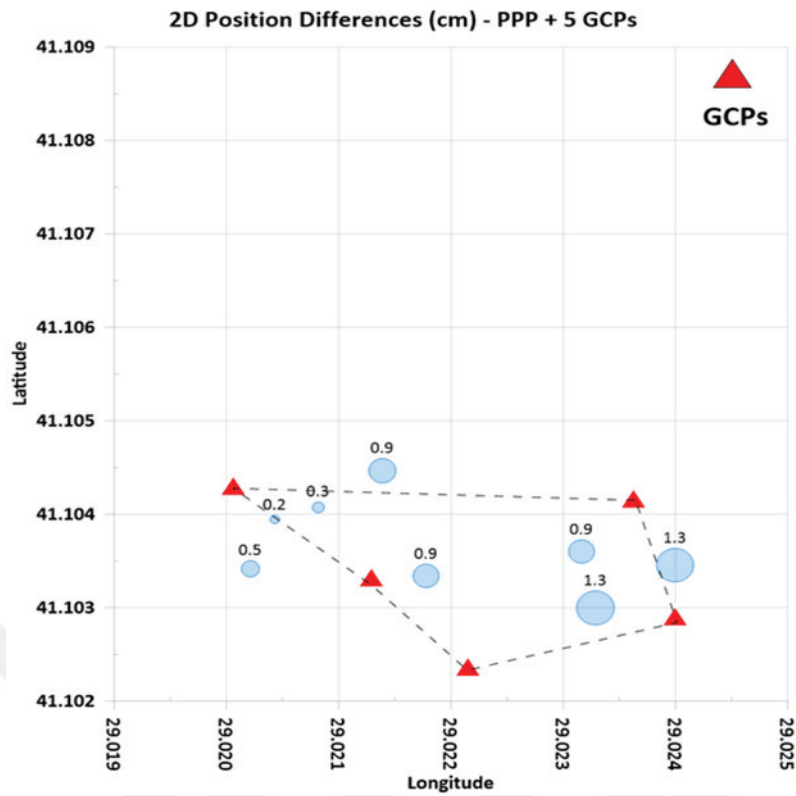


Figure A.14 : Horizontal differences between static GNSS and PPP + 5 GCPs model outputs

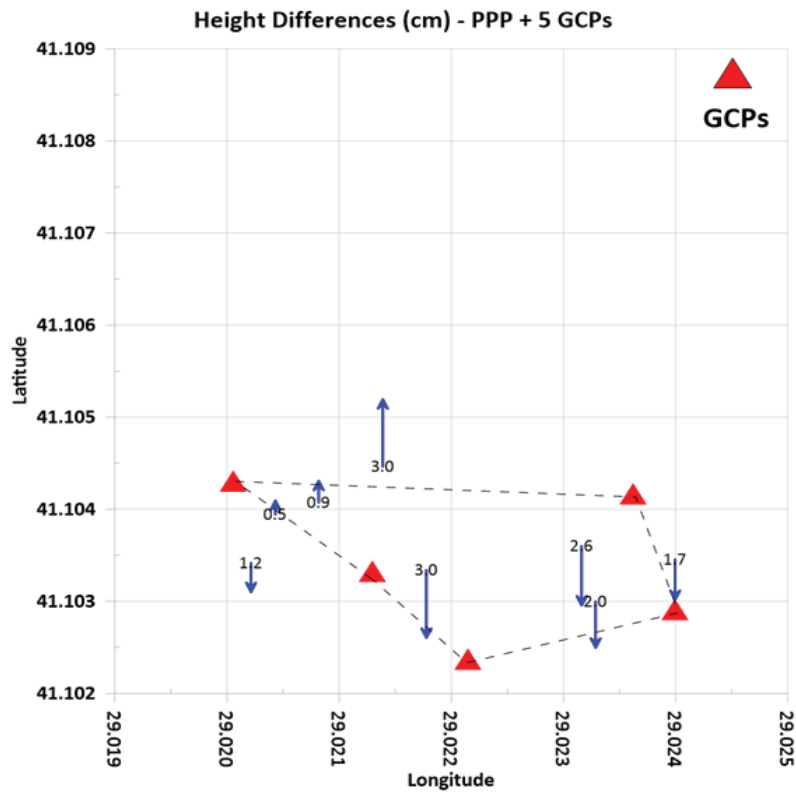


Figure A.15 : Vertical differences between static GNSS and PPP + 5 GCPs model outputs

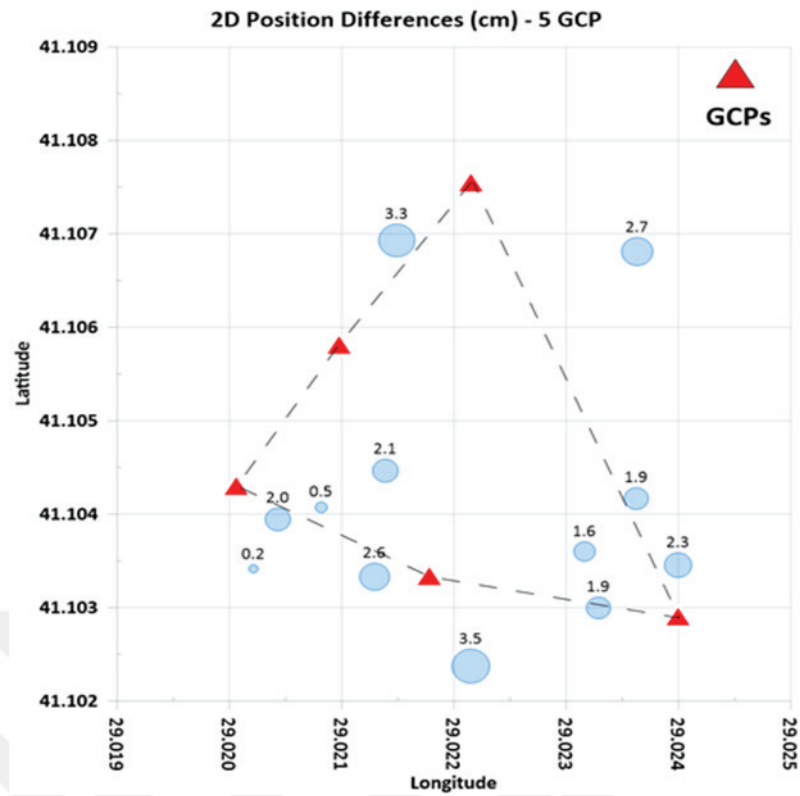


Figure A.16 : Horizontal differences between static GNSS and 5 GCPs model outputs

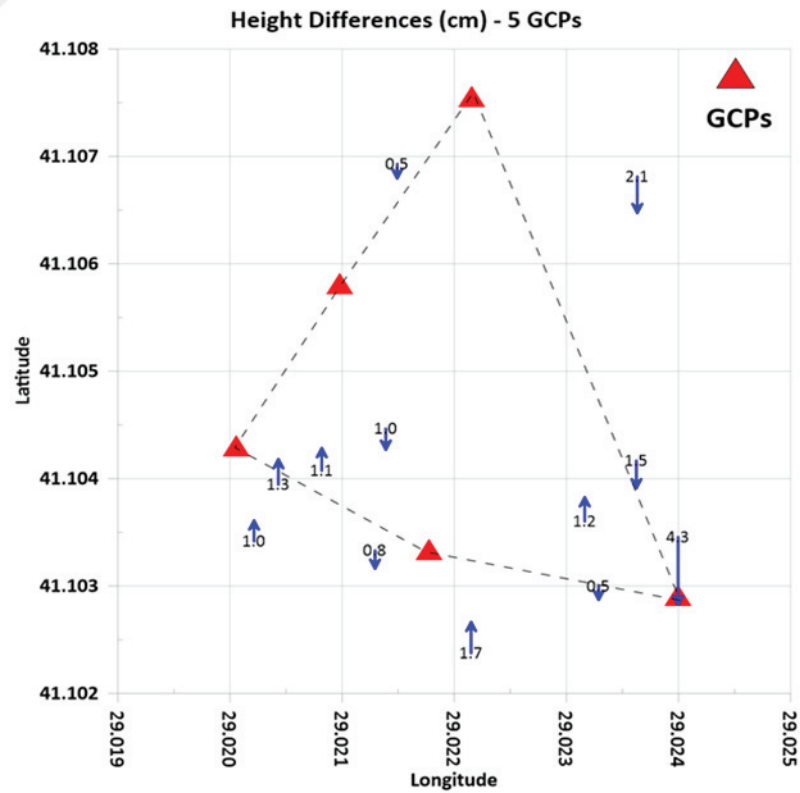


Figure A.17 : Vertical differences between static GNSS and 5 GCPs model outputs



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- ✓ Jan 2016 - Feb 2016, IHI Co. (Izmit Bay Crossing Suspension Bridge), Kocaeli (Turkey), Intern.
- ✓ Jun 2015 - Jul 2015, Nokia Here Maps, Istanbul (Turkey), Intern.
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PUBLICATIONS

Turan, E., Erol, B., Kuçak, R. A., & Erol, S. (2019). *Accuracy Assessment of Satellite Positioning Techniques on Unmanned Aerial Vehicle (UAV) Photogrammetry* Paper presented at the XXIX INTERNATIONAL SYMPOSIUM ON MODERN TECHNOLOGIES, EDUCATION AND PROFESSIONAL PRACTICE IN GEODESY AND RELATED FIELDS, Istanbul, Turkey. <http://www.gravity.itu.edu.tr/symp2019/>