Palestine Polytechnic University

College of Engineering

Civil & Architectural Engineering Department

Surveying and Geomatics Engineering

Graduation Project

Connecting the Palestinian Coordinates System to ITRF in Hebron District

Salem Abu-zneed **Saleh Al-Sabateen**

Husam Abu-Latifa **Mu'men Al-Jaidi**

Supervisor:

DR. Ghadi Zakarneh

Hebron-West Bank

Palestine

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ABSTRACT

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This project aims to connect the Palestinian coordinates system (Palestine 1923 Grid) in the area of Hebron district to an International Terrestrial Reference Frame (ITRF). This is required as new GPS/GNSS systems, technologies and algorithms enabled the use of baseline measurement in very long distances. Currently, there are hundreds of GNSS points measuring continuously and providing the raw data of the GNSS observations and their adjusted coordinates over the Internet. These points will be used for the connection between the Palestinian coordinates system and the selected ITRF.

The project applies the connection in the Hebron district. In this area, a group of the original triangulation points (Trigs) from the Palestinian geodetic network with their original easting and northing coordinates (E,N) are going to be reference for this project. These points will be used to build a 3D network using the measured baselines. A least squares solution is applied to calculate the geographic (λ, ϕ, h)/geocentric coordinates (X,Y,Z) in the selected ITRF system. These points must be measured over different period to monitor the movement and their stability in the ITRF system.

Finally, the relation between the Palestinian system and the selected ITRF system must be defined. This is applied by applying 3D coordinate transformation. The errors and differences between the two systems are going to be introduced and analyzed at the end of the project.

ملخص

ربط نظام الإحداثيات الفلسطينية في محافظة الخليل بنظام الإحداثيات الأرضي العالمي

سالم ابوزنيد صالح السباتين

مؤمن الجعيدي حسام ابولطيفة

المشرف:

الدكتور غادي زكارنة.

الهدف من هذا المشروع هو ربط نظام الإحداثيات الفلسطينية في محافظة الخليل Palestinian 1923 (Grid) مع نظام الإحداثيات الأرضي العالمي (ITRF). ونحتاج في هذا المشروع لاستخدام أنظمة GNSS /GPS بتقنياتها وأساليبها الحديثة,والتي تمكننا من قياس المسافات والخطوط بين النقاط يصل مداها لمسافات طويلة تصل الى مئات والاف الكيلومترات .وحاليا يوجد المئات من النقاط في نظام GNSS/GPS والتي تأخذ القراءات بشكل مستمر حيث تتوفر قراءات هذه النقاط وإحداثياتها المضبوطة على الانترنت . يتم استخدام هذه النقاط للربط بين نظام الاحداثيات الفلسطيني ونظام الاحداثيات العالمي ITRF.

في هذا المشروع يتم ربط أنظمة الإحداثيات لمنطقة الخليل في فلسطين. حيث يتم استخدام نقاط الشبكات المثلثية الجيودويسية ,سوف يتم استخدام هذه النقاط لتكون النقاط المرجعية في عملية ربط أنظمة الإحداثيات . كما سيتم بناء شبكة جيودوسية ثلاثية الأبعاد بالاعتماد على هذه النقاط وسيتم احتساب الإحداثيات الجغرافية أو الإحداثيات المركزية حسب النظام العالمي . وتتم القراءات على فترات مختلفة لتحديد ثبات هذه النقاط أو تحركها بالنسبة للنظام العالميITRF .

وأخيرا يجب تعريف العلاقة بين نظام الإحداثيات الفلسطينية ونظام الإحداثيات الأرضي الدولي. وهذا يطبق عن طريق اجراء نظام تحويل ثلاثي للأبعاد,الأخطاء الناتجة التحويل بين النظامين سوف يتم حسابها وتحليلها بنهاية المشروع.

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CHAPTER ONE

INTRODUCTIN

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1.1 Background

Terrestrial Reference System (TRS) is a spatial reference system, in such a system, positions of points attached to the solid surface of the Earth have coordinates which undergo only small variations with time, due to geophysical effects (tectonic or tidal deformations). A Terrestrial Reference Frame (TRF) is a set of physical points with precisely determined coordinates in a specific coordinate system (Cartesian, geographic, mapping...) attached to a Terrestrial Reference System. Such a TRF is said to be a realization of the TRS.

An International Terrestrial Reference Frame (ITRF) is a realization of the ITRS. New ITRF solutions are produced every few years, using the latest mathematical and surveying techniques to attempt to realize the ITRS as precisely as possible. Due to experimental error, any given ITRF will differ very slightly from any other realization of the ITRF. Also, the difference between the latest WGS84 and the latest ITRF is only a few centimeters.

The history of the ITRF goes back to 1984, when for the first time a combined TRF (called BTS84), was established using station coordinates derived from VLBI, LLR, SLR and Doppler/ TRANSIT (the predecessor of GPS) observations, 10 versions of the ITRF were published, starting with ITRF88 and ending with ITRF2000, each of which superseded its predecessor.

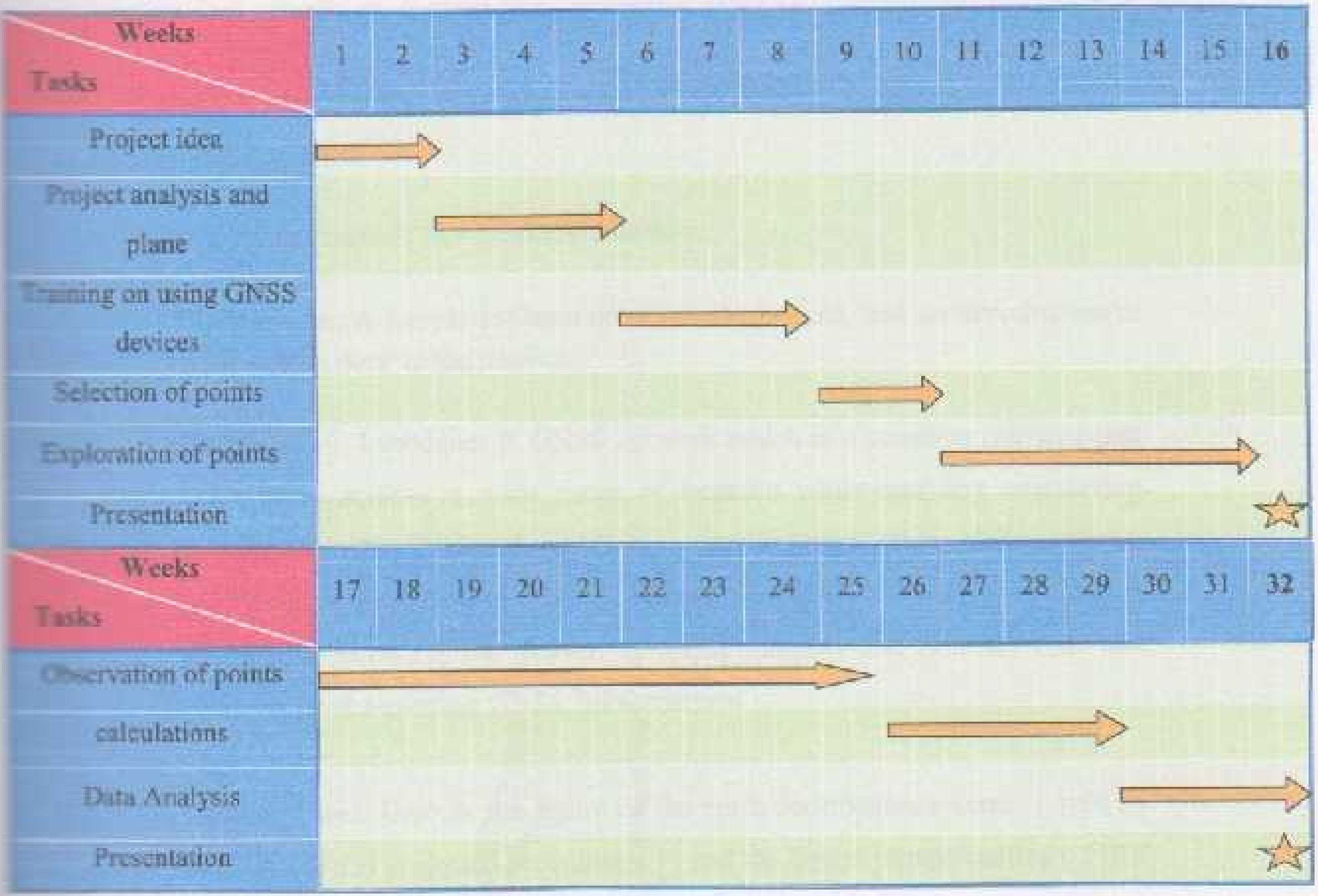
1.2 Objective

This project aim to convert from the Palestinian coordinates system to the international terrestrial reference frame ITRF, By monitoring points using the GPS, so that these points is a Trigg's, then we make a network of triangles between these points, after that we convert from Palestinian Coordinates system to ITFR system .

1.3 Time Table:

The time schedule shows the stages of developing in our work and the process of project growth that include Project determination, studying, collecting data, designing the entire system. Table (1-1) shows the first semester project growth. All tasks are referred to the theoretical background and the whole system analysis.

Table (1-1)Time Schedule for first semester



Weeks / Tasks	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Project idea	→	→														
Project analysis and plane			→	→	→											
Training on using GNSS devices						→	→	→								
Selection of points									→	→						
Exploration of points											→	→	→	→	→	→
Presentation																☆
Weeks / Tasks	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**	**32**
Observation of points	→	→	→	→	→	→	→	→	→							
calculations										→	→	→	→			
Data Analysis														→	→	→
Presentation																☆

In the first two weeks of the project have been identified in the project idea, and then been working on the analysis of the project and the analysis continued for three weeks, and then have been training on the use of devices Genesis and identify points and was for a period of five weeks, and then the process of exploration of point and its takes five weeks , after that we started Monitoring points that have been identified where the duration of the work at this stage takes twelve week .

1.4 Methodology

Method of working on this project will be by observed several points using the GPS then we Install the observed points after that we make a network coordinates by applying the coordinates of observed points , then we connect this network to the international terrestrial frame ITRF , after taking the ITRF coordinates applying it on the Palestinian Trigg's (Trigg's in West Bank which taken from the Hebron municipality) , finally we convert this coordinates Trigg's to international Coordinates .

1.5 Project Scope

This project consists of five chapters as follows:

- Chapter One: A simple explanation about the project, and an introduction to what will be done in the project.
- Chapter Two: Introduces A GNSS network which is a common resource that helps users achieve a wide range of benefits while enabling cost-saving solutions in the fields of surveying, mapping and other high accuracy positioning work. Whether you're building a new GNSS network or expanding an existing network, our proven solutions simplify the process, cost and complexity of deploying GNSS Infrastructure.
- Chapter Three: Discuss the figure of the earth coordinates system (type of coordinates and projected coordinates) , and the datum transformation (ITRF coordinates) .
- Chapter four: Least Square For GNSS Network.
- Chapter five: Fieldwork.
- Chapter six: the results of calculations that involves the ITRF coordinates and Palestinian coordinates and the transformation parameters.
- Chapter seven: conclusion and analysis of the results

CHAPTER TWO

GLOBAL NAVIGATION SATELLITE SYSTEM

2.1 Introduction.

2.2 Definition of the GNSS.

2.3 Global Navigation Satellite Systems.

2.4 GNSS Segment.

2.5 The Principle of GNSS positioning

2.6 GNSS Signal

2.7 GNSS Errors and Biases

2.8 GNSS Position Modes

2.9 GNSS Reference System

2.1 Introduction

Since earliest times, the human have interest to determine his position and his location with respect to other locations. He developed many methods to do that and he also used the sun and the stars to help him to determine his position. The oldest he used was the stars to determine his position with respect to the position of the stars this method give us an approximate location not the true location. Today with live in the era of precision we need to determine the position with high accuracy; so the human was needed to develop other methods that give us the needed accuracy so he send satellites to the space and developed them to help him in the positioning of his place.

2.2 Definition of the GNSS

Global Navigation Satellite System is a system used for positioning, tracking, and mapping in most cases is mentioned as synonymous with navigation; GNSS is the means that has translated the theoretical concept of navigation into an actual system, a quite friendly receiver, a commonly accepted and increasingly needed service.

In the past it was named Global Position System (GPS) which was developed by the US Military to allow the soldiers to autonomously determine their position within 10 to 20 meters accuracy without any other radio (or otherwise) communications.

Global coverage for the system is generally achieved by a satellite constellation of 20–30 medium Earth orbit (MEO) satellites spread between several orbital planes. The actual systems vary, but use orbital inclinations of >50° and orbital periods of roughly twelve hours (at an altitude of about 20,000 kilometers (12,000 mi)).

2.3 Global Navigation Satellite Systems

Different countries have developed that satellite navigation, the global system are, as shown in table (2-1):

1 GPS: The Global Positioning System (GPS) is a satellite-based navigation system that was developed by the U.S. Department of Defense (DoD) in the early1970s.

2 GLONASS is an all-weather global navigation satellite system developed by Russia. The GLONASS satellite system has much in common with the GPS system.

3 Galileo is a satellite-based global-navigation system proposed by Europe. Galileo is a civil-controlled satellite system to be delivered through a public-private partnership.

4 China has recently launched two domestically built navigation satellites, which form the first generation of a satellite-based navigation system. It is an all-weather regional navigation system, which is known as the Beidou Navigation System.

The satellites are placed in geostationary orbits at an altitude of approximately 36,000 km above the Earth's surface. The primary use of the system is in land and marine transportation.

Table (2-1) Global Navigation Systems

System	GPS	GLONASS	Galileo
Political entity	United States	Russian Federation	European Union
Coding	CDMA	FDMA/CDMA	CDMA
Orbital height	20,180 km(12,540 mi)	19,130 km (11,890 mi)	23,220 km (14,430 mi)
Period	11.97hours(11 h 58 m)	11.26hours(11 h 16 m)	14.08 hours (14 h 5 m)
Evolution per sidereal day	2	17/8	17/10
Number of satellites	At least 24	31, including,24 operational, 1 in preparation, 2 on maintenance, 3 reserve 1 on tests	4 test bed satellites in orbit, 22 operational satellites budgeted
Frequency	1.57542 GHz (L1 signal) 1.2276 GHz (L2 signal)	Around 1.602 GHz (SP) Around 1.246 GHz (SP)	1.164–1.215 GHz (E5a and E5b) 1.260–1.300 GHz (E6) 1.559–1.592 GHz (E2-L1-E1)
Status	Operational	Operational, CDMA in preparation	In preparation

2.4 GNSS Segment

GNSS consist of three distinct segments as shown in figure (2-1):

1. The space segment, the satellite or space vehicles.
2. The control segment, the ground tracking and monitoring stations.
3. The user segment, all users and there GNSS receivers.

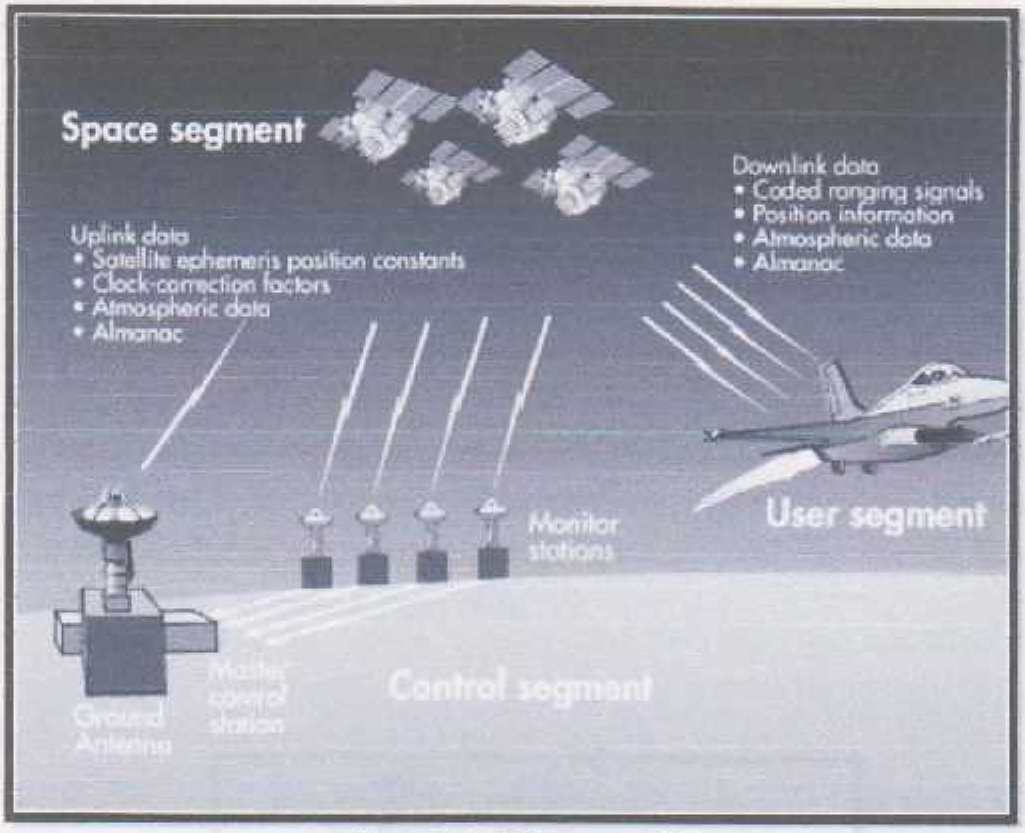


Figure (2-1): GNSS segments[3]

4.2.1 Space Segment

GNSS uses a constellation of satellites, each transmitting a composite ranging signal that includes a navigation message. The latter contains the information required to determine the coordinates of the satellites and bring the satellite clocks in line with the GNSS time.

Facts about GNSS

1. Each satellite weighs approximately 900 kilograms and is about five meters wide with the solar panels fully extended.
2. The base size of the constellation includes 21 operational satellites with three orbiting backups, for a total of 24.
3. They are located in six orbit satellites approximately 20,200 kilometers altitude. Each of the six orbits is inclined 55 degrees up from the equator, and is spaced 60 degrees apart, with four satellites located in each orbit.
4. The orbital period is 12 hours, meaning that each satellite completes two full orbits each 24-hour day.

4.2.2 Control Segments

Monitoring of the GNSS satellites, through checks of their operational health and determining their positions in space, is carried out by the operational control segment (OCS), As an example figure (2-2) show the control segments of the GPS. In particular, the segment takes care of: maintaining the satellites in due orbit through small maneuvers; introducing corrections and adjustments to satellite clocks and payload; tracking the GNSS satellites and uploading navigation data to each satellite of the constellation; and providing through commands major relocations in case of satellite failure. As shown in figure (2-3).

Figure (2-2): GPS control segment [4]

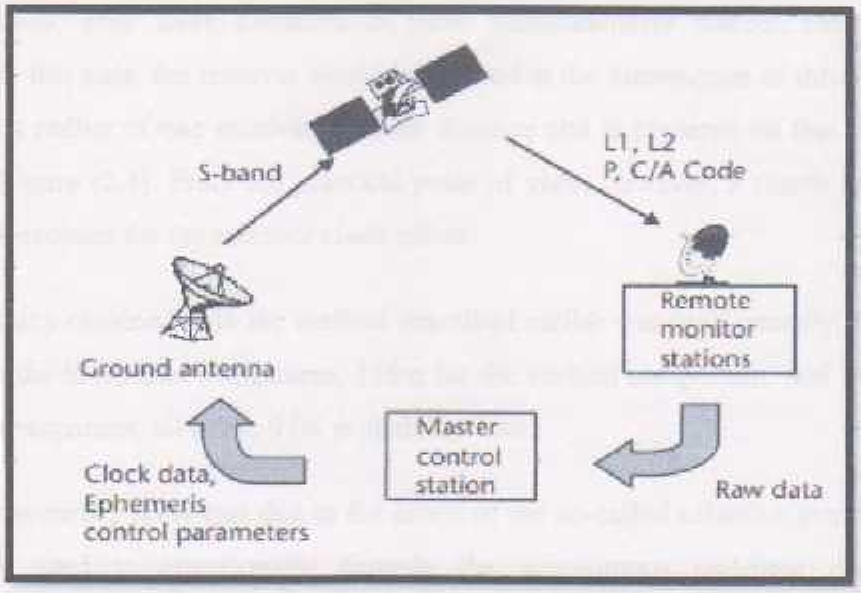


Figure (2-3): Basic structure and data flow of the GNSS control segment [2]

4.2.3 User segment

The user segment includes all military and civilian users. With a GNSS receiver connected to a GNSS antenna, a user can receive the GNSS signals, which can be used to determine his or her position anywhere in the world. GNSS is currently available to all users worldwide for free.

2.5 The Principle of GNSS positioning

The idea behind GNSS is rather simple. If the distances from a point on the Earth (a GNSS receiver) to three GNSS satellites are known along with the satellite locations, then the location of the point (or receiver) can be determined by simply applying the well-known concept of resection.

As mentioned before, each GNSS satellite continuously transmits a microwave radio signal composed of two carriers, two codes, and a navigation message. When a GNSS receiver is switched on, it will pick up the GNSS signal through the receiver antenna. Once the receiver acquires the GNSS signal, it will process it using its built-in software. The partial outcome of the signal processing consists of the distances to the GNSS satellites through the digital codes (known as the pseudoranges) and the satellite coordinates through the navigation message.

Theoretically, only three distances to three simultaneously tracked satellites are needed. In this case, the receiver would be located at the intersection of three spheres; each has a radius of one receiver-satellite distance and is centered on that particular satellite Figure (2.4). From the practical point of view, however, a fourth satellite is needed to account for the receiver clock offset.

The accuracy obtained with the method described earlier was until recently limited to 100m for the horizontal component, 156m for the vertical component, and 340 ns for the time component, all at the 95% probability level.

This low accuracy level was due to the effect of the so-called selective availability, a technique used to intentionally degrade the autonomous real-time positioning accuracy to unauthorized users. With the recent presidential decision of terminating the selective availability, the obtained horizontal accuracy is expected to improve to about 22m (95% probability level). To further improve the GNSS positioning accuracy, the so-called differential method, which employs two receivers simultaneously tracking the same GNSS satellites, is used. In this case, positioning accuracy level of the order of a subcentimeter to a few meters can be obtained.

Other uses of GNSS include the determination of the user's velocity, which could be determined by several methods. The most widely used method is based on estimating the Doppler frequency of the received GNSS signal. It is known that the Doppler shift occurs as a result of the relative satellite-receiver motion.

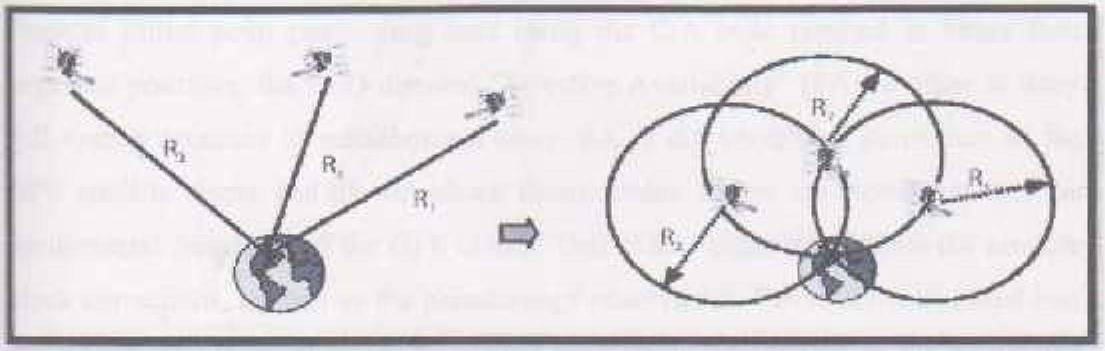


Figure (2-4): Basic idea of GNSS positioning [6]

- **Calculating the distance to the satellite**

$$R = V \times T \tag{2.1}$$

Where:

R: Distance.

V: Basic idea of GNSS positioning 300,000 kilometers per second.

T: Time in transit.

2.6 GNSS Signals

Each GPS satellite transmits data on two frequencies, L1 (1575.42 MHz) and L2 (1227.60 MHz). The atomic clocks aboard the satellite produces the fundamental L-band frequency, 10.23 Mhz. The L1and L2 carrier frequencies are generated by multiplying the fundamental frequency by 154 and 120, respectively, as shown in table(2-2). Two pseudorandom noise (PRN) codes, along with satellite ephemerides (Broadcast Ephemerides), ionospheric modeling coefficients, status information, system time, and satellite clock corrections, are superimposed onto the carrier frequencies, L1 and L2. The measured travel times of the signals from the satellites to the receivers are used to compute the pseudoranges.

The Course-Acquisition (C/A) code, sometimes called the Standard Positioning Service (SPS), is a pseudorandom noise code that is modulated onto the L1 carrier. Because initial point positioning tests using the C/A code resulted in better than expected positions, the DoD directed "Selective Availability" (SA) in order to deny full system accuracy to unauthorized users. SA is the intentional corruption of the GPS satellite clocks and the Broadcast Ephemerides. Errors are introduced into the fundamental frequency of the GPS clocks. This clock "dithering" affects the satellite clock corrections, as well as the pseudorange observables. Errors are introduced into the Broadcast Ephemerides by truncating the orbital information in the navigation message.

The Precision (P) code, sometimes called the Precise Positioning Service (PPS), is modulated onto the L1 and L2 carriers allowing for the removal of the first order effects of the ionosphere. The P code is referred to as the Y code if encrypted. Y code is actually the combination of the P code and a W encryption code and requires a DoD authorized receiver to use it. Originally the encryption was intended as a means to safe-guard the signal from being corrupted by interference, jamming, or falsified signals with the GPS signature. Because of the intent to protect against "spoofing," the encryption is referred to as "Anti-spoofing" (A-S). A-S is either "on" or it's "off;" there is no variable effect of A-S as there is with SA.

Table (2-2): GNSS Signal Codes and Carrier Frequencies

Carrier L_band		**Codes**		**Satallite Massege**
		Civilian C/A-code	Malitriy PY-code	
L1	1575.42 Mhz 19cm wavelength	Present 293 m wavelengh	Present 29.3 m wavelength	User messages Satellite constants Satellite positions
L2	1227.60 MHz 24cm wavelength	Not present	Present 29.3 m wavelength	

2.7 GNSS Errors and Biases

The GNSS mesurments may be affected by many error and baises this error can be classified in four groupes they are listed in Figure (2.5).

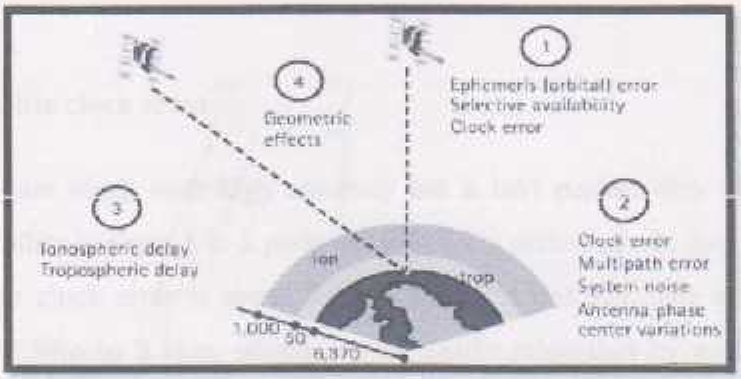


Figure (2-5): GNSS errors and biases [1]

1. The errors originating at the satellites:
    - Ephemeris or orbital error.
    - Selective availability.
    - Satellite clock error
2. The errors originating at the receiver:
    - Receiver clock error.
    - Multipath error.
    - Receiver noise.
    - Antenna phase center variations.
3. The signal propagation errors:
    - Ionospheric delay.
    - Tropospheric delay.
4. The Geometric effects.

2.7.1 Selective Availability (Anti Spoofing)

GNSS was originally designed so that real-time autonomous positioning and navigation with the civilian C/A code receivers would be less precise than military P-code receivers. Surprisingly, the obtained accuracy was almost the same from both receivers. To ensure national security, the U.S. DoD implemented the so-called selective availability (SA) on Block II GPS satellites to deny accurate real-time autonomous positioning to unauthorized users. SA was officially activated on March 25, 1990.

2.7.2 Satellite clock error

GNSS satellite use clock with high accuracy but it isn't perfect they include some error. Their stability is about 1 to 2 parts in 10^{13} over a period of one day. This means that the satellite clock error is about 8.64 to 17.28 ns per day. The corresponding range error is 2.59m to 5.18m, which can be easily calculated by multiplying the clock error by the speed of light (299,729,458 m/s).

2.7.3 Receiver measurments noise

The receiver measurement noise results from the limitations of the receiver's electronics. Generally, a GPS receiver performs a self-test when the user turns it on. However, for high-cost precise GPS systems, it might be important for the user to perform the system evaluation. Two tests can be performed for evaluating a GPS receiver (system):

1. Zero baseline test.
2. Short baseline test.

2.7.4 Ionosphere and troposphere refraction

At the uppermost part of the earth's atmosphere, ultraviolet and X-ray radiations coming from the sun interact with the gas molecules and atoms. These interactions result in gas ionization: a large number of free "negatively charged" electrons and "positively charged"" atoms and molecules. Such a region of the atmosphere where gas ionization takes place is called the ionosphere. It extends from an altitude of approximately 50 km to about 1,000 km or even more, as shown in figure(2-6).

The troposphere is the electrically neutral atmospheric region that extends up to about 50 km from the surface of the earth. The troposphere is a not dispersive medium for radio frequencies below 15 GHz.

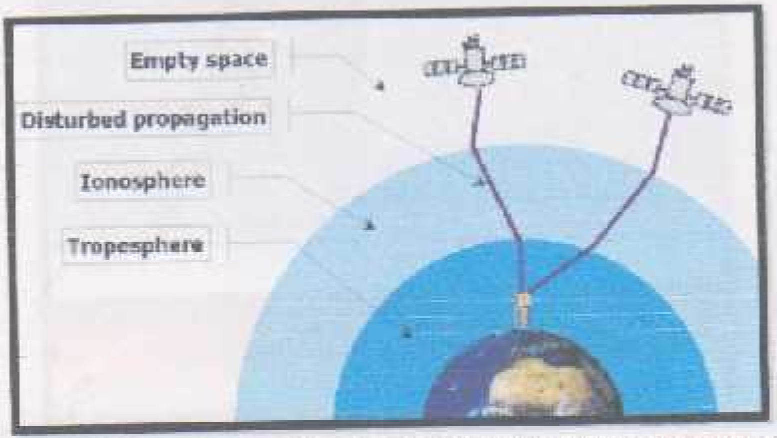


Figure (2-6): Influenced propagation of radio waves through the earth's atmosphere[3]

Both ionosphere and troposphere cause bending of the signals. This bending of radio waves is called refraction. The problem with the Ionosphere is the electrically charged particles that drag on the incoming signal. In the troposphere, the problem is with the water vapor content which does the same thing. These problems are even further exacerbated when a satellite is low on the horizon. This is because a line tangent to the surface of the Earth (or nearly so) passes through a much thicker layer of atmosphere than if that line were pointing straight up.

To deal with refractions the satellite's NAV-massage includes an atmospheric refraction model that compensates for as much as 50-70% of the error and to use a dual-frequency receiver which simultaneously collects the signals on both the Ll and L2 carriers. Because the amount of refraction that a radio wave experiences is inversely proportional to its frequency, using two different frequencies transmitted through the same atmosphere at the same time makes it relatively easy to compute the amount of refraction taking place and compensate it.

2.7.5 Mask Angle

cut-off angle: The point above the observer's horizon below which satellite signals are no longer tracked and/or processed. 15° to 25° is typical,as shown in figure (2-7).

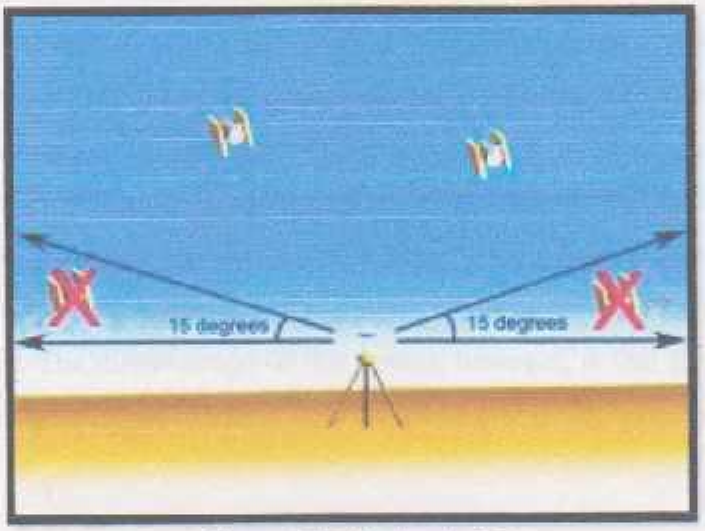


Figure (2-7): Mask angle [5]

2.7.6 Multi path Error

Multipath error occurs when the GPS signal arrives at the receiver antenna through different paths. These paths can be the direct line of sight signal and reflected signals from objects surrounding the receiver antenna see Figure(2.8).

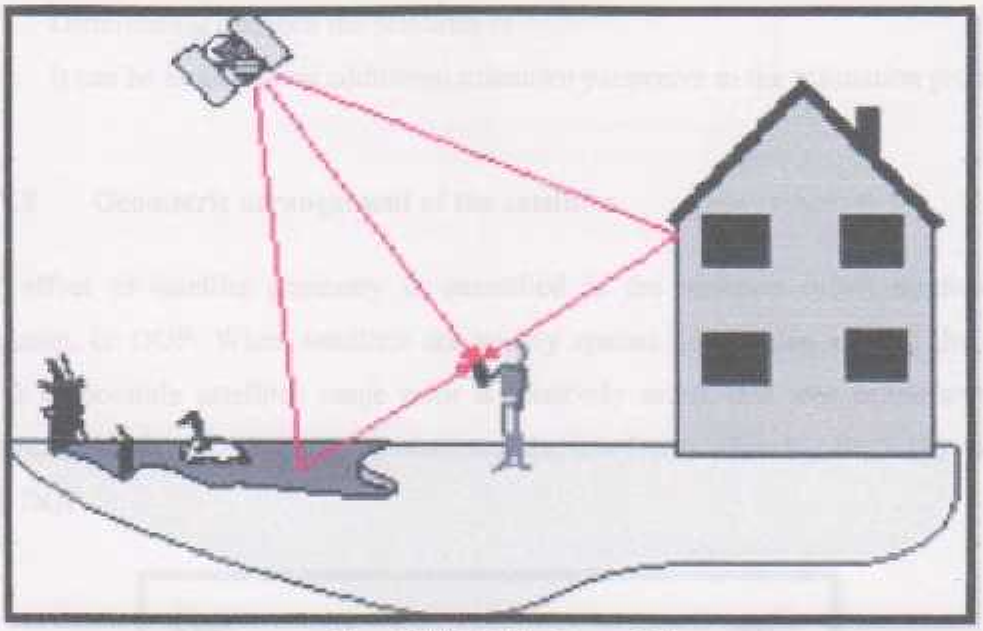


Figure (2-8): Multi path error [4]

There are several options to reduce the effect of multipath:

1. The straightforward option is to select an observation site with no reflecting objects in the vicinity of the receiver antenna.
2. Another option to reduce the effect of multipath is to use a chock ring antenna (a chock ring device is a ground plane that has several concentric metal hoops, which attenuate the reflected signals).
3. As the GNSS signal is right-handed circularly polarized while the reflected signal is left-handed, reducing the effect of multipath may also be achieved by using an antenna with a matching polarization to the GNSS signal (i.e., right-handed). The disadvantage of this option, however, is that the polarization of the multipath signal becomes right-handed again if it is reflected twice.

2.7.7 Reciver Clock error

GNSS reciever use inexpensive crystal clocks, which are much less accurate than the satellite clocks. As such, the receiver clock error is much larger than that of the GNSS satellite clock. It can, however, be removed through

1. Differencing between the satellites or
2. It can be treated as an additional unknown parameter in the estimation process.

2.7.8 Geometric arrangement of the satellites

The effect of satellite geometry is quantified in the measure called dilution of precision, or DOP. When satellites are widely spaced the overlap area of the two zones of possible satellites range error is relatively small, this area called area of positional ambiguity. Figure (2-9) illustrates the low DOP, while figure (2-10) shows high DOP.

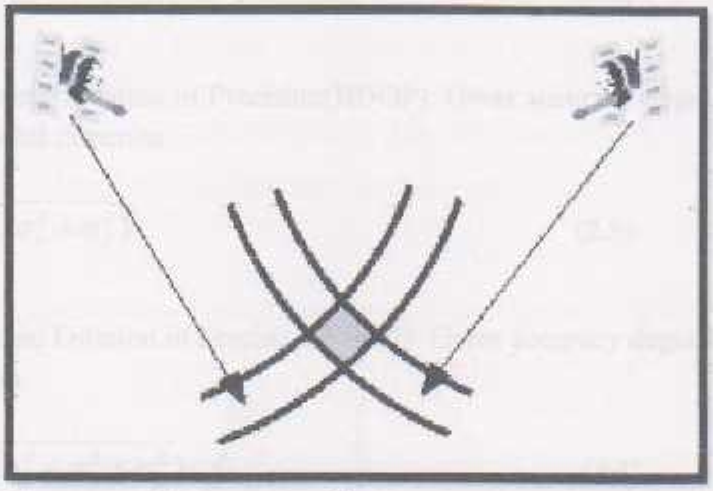


Figure (2-9): Well spaced satellites Low uncertainty of position [3]

The best way to minimize the effect of DOP is to observe as many satellites as possible. And these are the values of dilution of precision:

1. A DOP value less than 2 is considered excellent.
2. A DOP value between 2 and 3 is considered very good.
3. A DOP value between 3 and 5 is considered good.
4. A DOP value greater than 5 and less than 6 is considered fair.

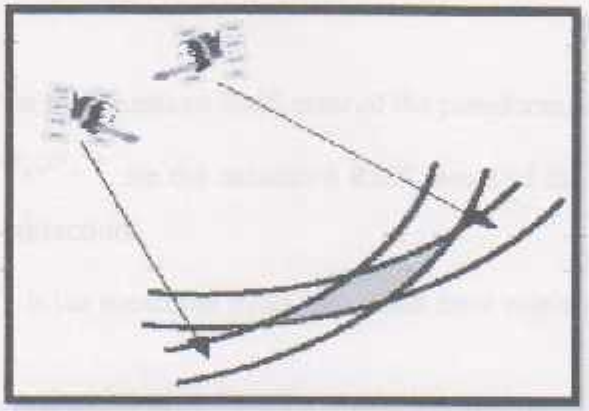


Figure (2-10): Poorly spaced satellites High uncertainty of position [3]

Different types of Dilution of Precision or DOP can be calculated depending on the dimension; these values are calculated by the covariance matrix of the position generated from least squares adjustment:

- Vertical Dilution of Precision(VDOP): Gives accuracy degradation in vertical direction.

$$VDOP= \frac{\sigma_z}{\sigma} \qquad (2.2)$$

- Horizontal Dilution of Precision(HDOP): Gives accuracy degradation in horizontal direction.

$$HDOP= \frac{1}{\sigma} * \sqrt{\sigma_x^2+\sigma_y^2}) \qquad (2.3)$$

- Positional Dilution of Precision(PDOP): Gives accuracy degradation in 3D position.

$$PDOP= \frac{1}{\sigma} * \sqrt{\sigma_x^2+\sigma_y^2+\sigma_z^2}) \qquad (2.4)$$

- Time dilution of precision(TDOP): Gives accuracy in time.

$$TDOP= \frac{\sigma_b}{\sigma} \qquad (2.5)$$

- Geometric Dilution of Precision(GDOP): Gives accuracy degradation in 3D position and time.

$$GDOP= \frac{1}{\sigma} * \sqrt{\sigma_x^2+\sigma_y^2+\sigma_z^2+\sigma_b^2}) \qquad (2.6)$$

Where:

$\sigma =$ is the measured RMS error of the pseudorange.

$\sigma_x, \sigma_y, \sigma_y =$ are the measured RMS errors of the user position in the xyz directions.

$\sigma_b =$ is the measured RMS user clock error expressed in distance.

2.8 GNSS Position Modes

Positioning with GPS can be performed by either of two ways: point positioning or relative positioning

2.8.1 GNSS Point Positioning

Involves only one GNSS receiver that is, one GNSS receiver simultaneously tracks four or more GPS satellites to determine its own coordinates with respect to the center of the Earth, as shown Figure (2-11). Almost all of the GNSS receivers currently available on the market are capable of displaying their point positioning coordinates.

To determine the receivers point position at any time, the satellite coordinates as well as a minimum of four ranges to four satellites are required.

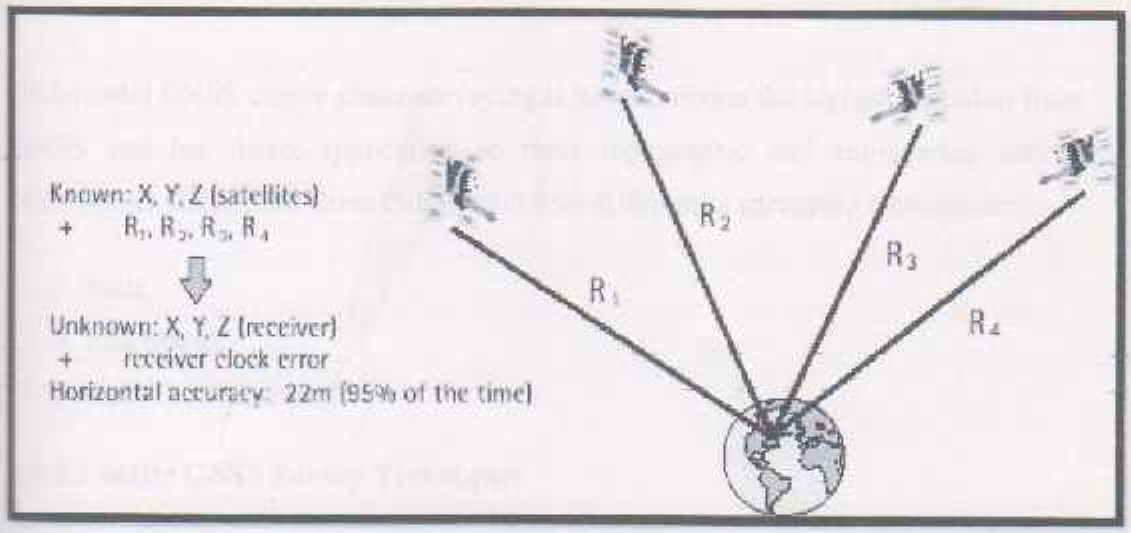


Figure (2-11): Principal of GNSS point

2.8.2 GNSS Relative Positioning

GNSS relative positioning, also called differential positioning, employs two GNSS receivers simultaneously tracking the same satellites to determine their relative coordinates, as shown Figure (2-12). Of the two receivers, one is selected as a reference, or base, which remains stationary at a site with precisely known coordinates. The other receiver, known as the rover or remote receiver, has its coordinates unknown. The rover receiver may or may not be stationary, depending on the type of the GNSS operation. A minimum of four common satellites is required for relative positioning.

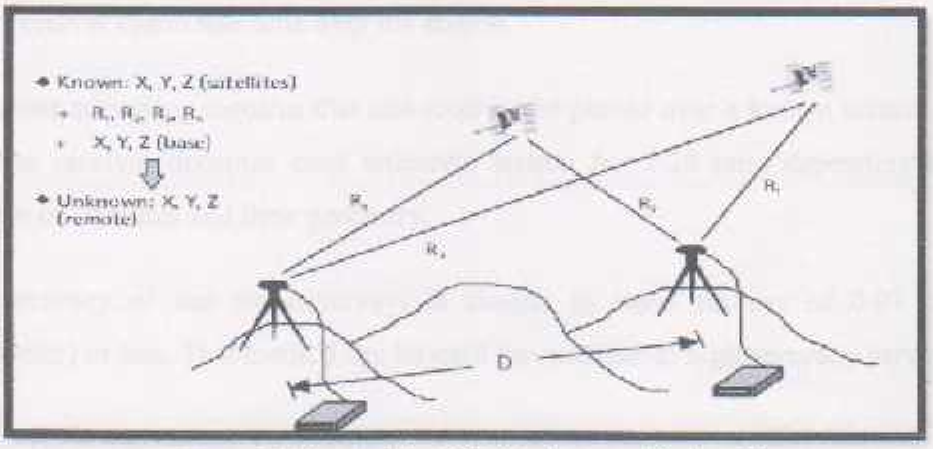


Figure (2-12): principle of GNSS relative positioning [3]

Differential GNSS carrier phase surveying is used to obtain the highest precision from GNSS and has direct application to most topographic and engineering survey activities. D GNSS uses three Different GNSS differential surveying techniques:

1. Static.
2. Fast Static.
3. Real Time Kinematic.

2.8.2.1 Static GNSS Survey Techniques

This was the first method to be developed for GNSS surveying. It can be used for measuring long baselines (usually 20km (16 miles) and over).

The base should placed over an point whose coordinates known with high accuracy and the rover will placed over an point whose coordinates are unknown. Both GNSS receivers must receive signals from the same four (or more) satellites for a period of time that can range from a few minutes to several hours, depending on the conditions of observation and precision required.

Static GNSS has the capability to produce relative positions at the sub-centimeter level on relatively short distances (a few hundred kilometers) and at the centimeter level over long distances (up to thousands of kilometers)

2.8.2.2 Fast Static GNSS Survey Techniques

This technique is similar to the static technique. The different between them that the rover receiver spend less time over the station.

Fast static surveying requires that one receiver be placed over a known control point. A rover receiver occupies each unknown station for 5-20 min, depending on the number of satellites and their geometry.

The accuracy of fast static surveys is similar to static surveys of 0.03 feet (1 centimeter) or less. This method can be used for medium-to high accuracy survey.

2.8.2.3 RTK Surveying Techniques

RTK stands for Real Time Kinematic. It is a Kinematic on the Fly survey carried out in real time. The Reference Station has a radio link attached and rebroadcasts the data it receives from the satellites.

The Rover also has a radio link and receives the signal broadcast from the Reference. The Rover also receives satellite data directly from the satellites via its own GNSS Antenna. These two sets of data can be processed together at the Rover to resolve the ambiguity and therefore obtain a very accurate position relative to the Reference receiver.

Once the Reference Receiver has been set up and is broadcasting data through the radio link, the Rover Receiver can be activated.

When it is tracking satellites and receiving data from the Reference, it can begin the initialization process. This is similar to the initialization performed in a post-processed kinematic on the fly survey, the main difference being that it is carried out in real-time.

Once the initialization is complete, the ambiguities are resolved and the Rover can record point and coordinate data.

RTK surveys can be accurate to within 0.05 to 0.10 feet (2– 3 centimeters), providing a good static network and calibration were performed prior to performing the RTK survey. As shown in figure (2-13).

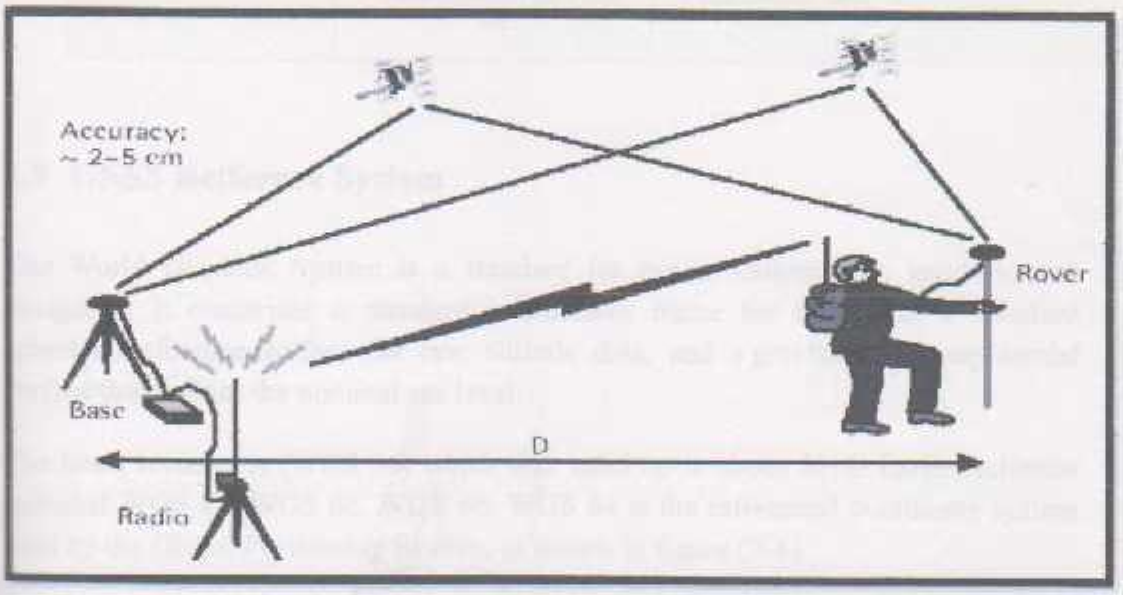


Figure (2-13): RTK GNSS Surveying [2]

Table (2-3) shows the requirement, application, and accuracy, for each type of relative GNSS position (Static, Rapid Static(Fast), and Real Time Kinematic).

Table (2-3) GNSS Relative Positioning

Concept	Requirements	Applications	Accuracy
Static (Post-processing)	•L1 or L1/L2 GNSS S receiver •computer for post-processing. •45 min to 1 hr minimum observation time	• Control surveys (that require high accuracy)	• Sub centimeter level
Rapid Static (Post-processing)	• L1/L2 GNSS receiver • 5-20 min observation time	• Control surveys (that require medium to high accuracy	• Sub centimeter level
Real Time Kinematic (Real-Time)	For post-processing: • L1/L2 GNSS receiver • Computer For real-time: • L1/L2 GNSS receiver • Internal or external processor (computers) • Radio/modem data link set	• Real-time high accuracy surveys • Location surveys • Medium accuracy control surveys • Photo control • Continuous topo	• Sub decimeter level

2.9 GNSS Reffernce System

The World Geodetic System is a standard for use in cartography, geodesy, and navigation it comprises a standard coordinates frame for the earth, a standard spherical reference surface for raw altitude data, and a gravitational equipotential surface that defines the nominal sea level.

The latest revision is (WGS 84) which was valid up to about 2010. Earlier schemes included WGS 72, WGS 66, WGS 60. WGS 84 is the referenced coordinate system used by the Global Positioning System, as shown in figure (2-4).

Table (2-4): parameter of WGS 84

Ellipsoidal name	Semi major axis (a in meters)	Semi minor axis (a in meters)
WGS 84	6378137	298.257223563

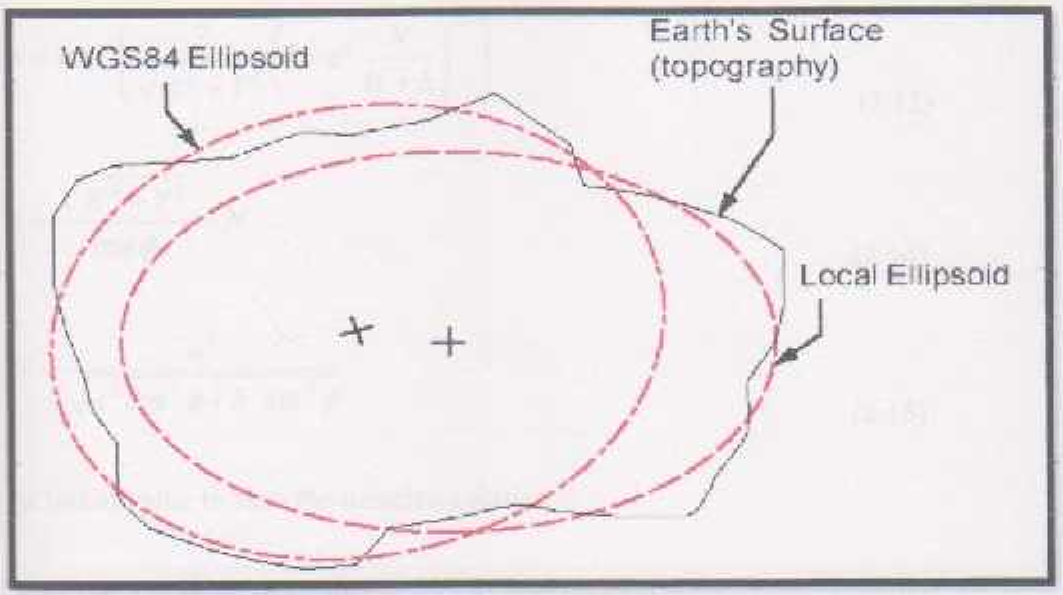


Figure (2-14): WGS 84 [1]

The other geometric parameters are computed using the following equations:

$$r = a(1+n^2/4)/(1+n) \quad (2.7)$$

$$n = f/(2-f) \quad (2.8)$$

$$e^2 = f(2-f) \quad (2.9)$$

$$e'^2 = e^2/(1-f)^2 \quad (2.10)$$

$$b = a(1-f) \quad (2.11)$$

The absolute positions obtained from GPS are based on the 3-D WGS84 ellipsoid. Coordinate outputs are on a Cartesian system(X-Y-Z) relative to WGS84 rectangular coordinate. These coordinate can be transformed to λ, ϕ, and h by an iterative solution where:

$$\lambda = \tan^{-1}\frac{Y}{X} \quad (2.12)$$

$$\phi = \tan^{-1}\left(\frac{Z}{\sqrt{X^2+Y^2}} \left(1 - e^2 \frac{N}{N+h} \right)^{-1} \right) \tag{2.13}$$

$$h = \frac{\sqrt{X^2+Y^2}}{\cos\phi} - N \tag{2.14}$$

$$N = \frac{a^2}{\sqrt{a^2\cos^2\phi + b\ \sin^2\phi}} \tag{2.15}$$

As initial value to start the iterative solution:

$$\phi = \tan^{-1}\frac{Z}{\sqrt{X^2+Y^2}}\left(1-e^2\right)^{-1} \tag{2.16}$$

The inverse problem to find the X, Y, and z, from λ, ϕ, and h;

$$X = (N+h)\cos\phi\cos\lambda \tag{2.17a}$$

$$Y = (N+h)\cos\phi\cos\lambda \tag{2.17b}$$

$$Z = \left(\left(1-e^2\right)N + h\right)\sin\phi \tag{2.17c}$$

These coordinates can be transformed to local datum system using 3D similarity transformation acording to the following equations:

X (Local) = X (WGS 84) + ΔX (2.18)

Y (Local) = Y(WGS 84) + ΔY (2.19)

Z (Clarke 1880) = Z (WGS 84) + ΔZ (2.20)

Where: ΔX = 230.00 m, ΔY = 71.00 m, ΔZ = -273 m

CHAPTER THREE

COORDINATES SYSTEMS

3.1 Introduction.

3.2 Conversion between positions coordinates systems

3.3 Coordinate Systems.

3.4 International Terrestrial Reference Frame

3.5 North Pole Movement.

3.6 Tectonic plate movement

3.1 Introduction:

A coordinate system is a set of rules that state the correspondence between coordinate and points; a coordinate is one of a set of N numbers individuating the location of a point in an N-dimensional space. A coordinate system is defined once a point known as origin, a set of N lines, called axes, all passing for the origin and having well-known relationships to each other, and a unit length are established.

In GNSS application, the position of a point in a coordinate system can be expressed in Figure(3-1).

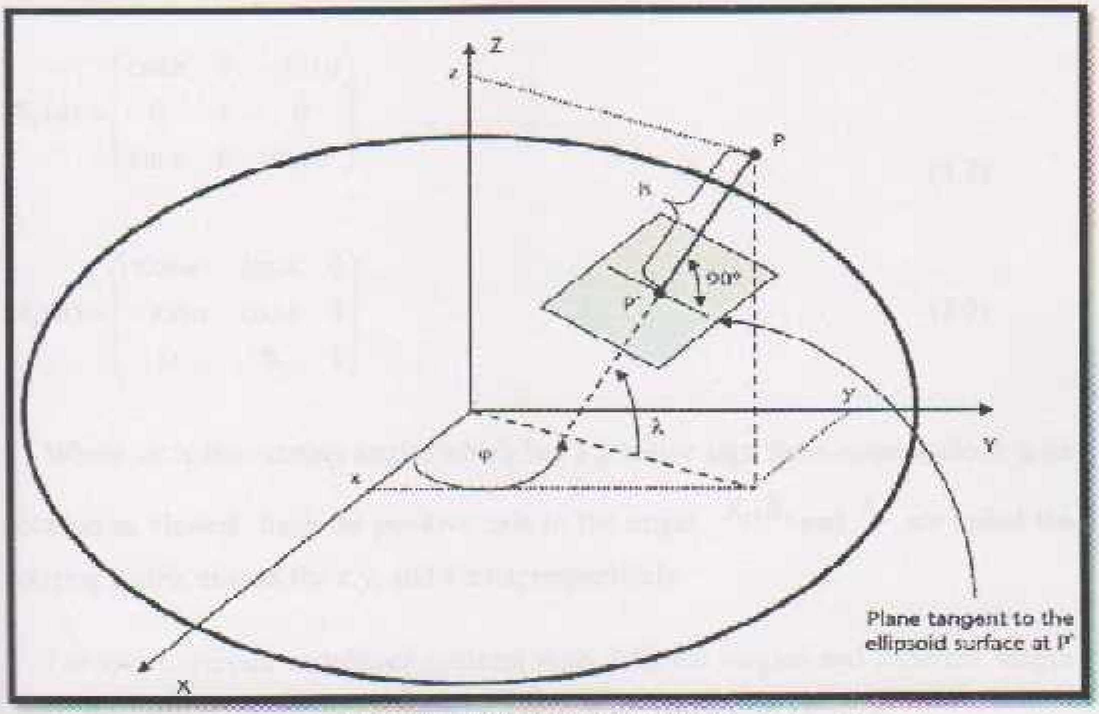


Figure (3-1): Geodetic coordinate [3]

- Cartesian coordinates (x, y, z);
- Ellipsoidal or geodetic (also called geographic) coordinates (λ, ϕ, h): λ is the latitude, ϕ is the longitude, and h is the height above the surface of the earth .

3.2 Conversion between position coordinates systems

Any Cartesian coordinate system can be transformed to another Cartesian coordinate system through three succeeded rotations if their origins are the same and if they are both right-handed or left-handed coordinate systems. These three rotational matrices are:

$$R_1(a)=\begin{pmatrix}1 & 0 & 0\\ 0 & \cos a & \sin a\\ 0 & -\sin a & \cos a\end{pmatrix} \tag{3.1}$$

$$R_2(a)=\begin{pmatrix}\cos a & 0 & -\sin a\\ 0 & 1 & 0\\ \sin a & 0 & \cos a\end{pmatrix} \tag{3.2}$$

$$R_3(a)=\begin{pmatrix}\cos a & \sin a & 0\\ -\sin a & \cos a & 0\\ 0 & 0 & 1\end{pmatrix} \tag{3.3}$$

Where a is the rotating angle, which has a positive sign for a counter-clock wise rotation as viewed from the positive axis to the origin. R_1, R_2 and R_3 are called the rotating matrix around the x,y, and z-axis, respectively.

For two Cartesian coordinate systems with different origins and different length units, the general transformation can be given in vector (matrix) form as

$$X_n = X_0 + \mu R X_{old} \tag{3.4a}$$

OR

$$\begin{pmatrix}x_n\\ y_n\\ z_n\end{pmatrix}=\begin{pmatrix}x_0\\ y_0\\ z_0\end{pmatrix}+\mu R\begin{pmatrix}x_{old}\\ y_{old}\\ z_{old}\end{pmatrix} \tag{3.4b}$$

Where µ is the scale factor (or the ratio of the two length units), and R is a transformation matrix that can be formed by three suitably succeeded rotations. x_n and x_{old} denote the new and old coordinates, respectively; x_0 denotes the translation vector and is the coordinate vector of the origin of the old coordinate system in the new one.

If rotational angle α is very small, then one has sin α ≈α and cos α ≈ 0. In such a case, the rotational matrix can be simplified. If the three rotational angles a_1, a_2, a_3 in R of Eq are very small then R can be written as:

$$R = \begin{pmatrix} 1 & a_3 & -a_2 \\ -a_3 & 1 & a_1 \\ a_2 & -a_1 & 0 \end{pmatrix} \tag{3.5}$$

Where: a_1, a_2, a_3 are small rotating angles around the x, y and z-axis, respectively.

3.3 Coordinate Systems

We have several coordinate systems here are the most important three systems are:

- Geographic coordinate system.
- Cartesian coordinate system.
- Top centric coordinate system.

3.3.1 Geographic Coordinat System

A geographic coordinate system is a coordinate system that enables every location on the Earth to be specified by a set of numbers or letters. The coordinates are often chosen such that one of the numbers represents vertical position, and two or three of the numbers represent horizontal position. A common choice of coordinates is latitude, longitude and elevation, as shown in figure (3-2).

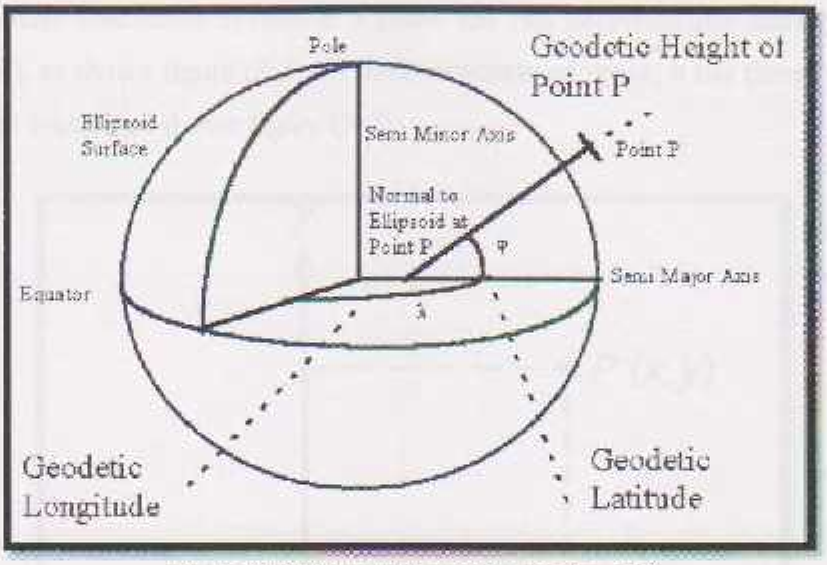


Figure (3-2): Geographic coordinate system [6]

The latitude (φ) of a point on the Earth's surface is the angle between the equatorial plane and a line that passes through that point and is normal to the surface of a reference ellipsoid which approximates the shape of the Earth.

The Longitude (λ) of a point on the Earth's surface is the angle east or west from a reference meridian to another meridian that passes through that point. All meridians are halves of great ellipses (often improperly called great circles), which converge at the north and south poles.

The geodetic (ellipsoid or normal) height (h) at a point is the distance from the reference ellipsoid to the point in the direction normal to the ellipsoid

3.3.2 Cartesian Coordinat system

A Cartesian coordinate system is a coordinate system that specifies each point uniquely in a plane by a pair of numerical coordinates, which are the signed distances from the point to two fixed perpendicular directed lines, measured in the same unit of length. Each reference line is called a coordinate axis or just axis of the system, and the point where they meet is its origin, usually at ordered pair (0, 0). The coordinates can also be defined as the positions of the perpendicular projections of the point onto the two axes, expressed as signed distances from the origin.

A Cartesian coordinate system in a plane has two perpendicular lines (the x-axis and y-axis), as shown figure (3-3); in three-dimensional space, it has three (the x-axis, y-axis, and z-axis), as shown figure (3-4).

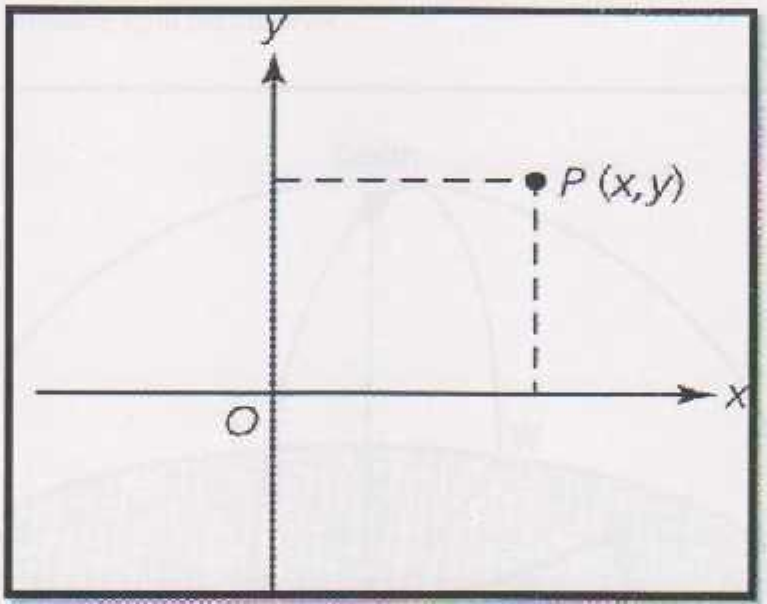


Figure (3-3): Two-dimensional space of Cartesian coordinate [9]

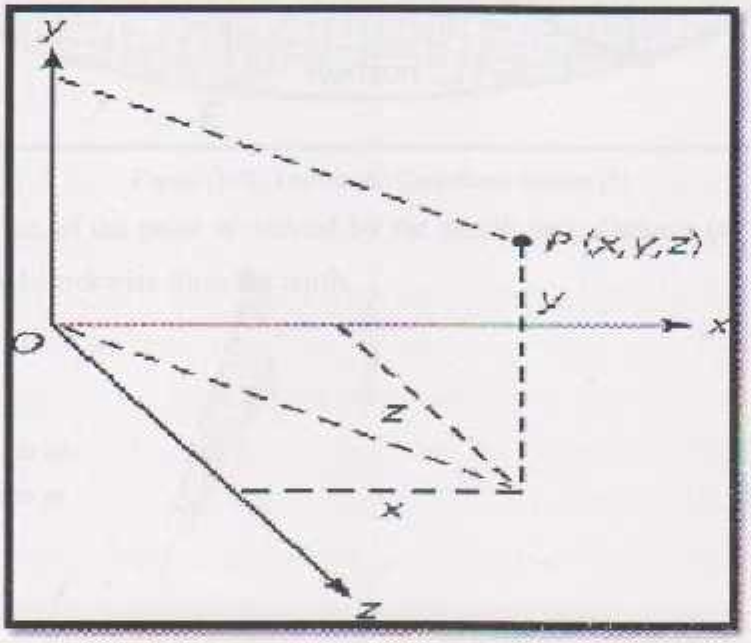


Figure (3-4): Three-dimensional space of Cartesian coordinate[9]

3.3.3 Topocentric Coordinat System

This system use for tracking purposes is illustrated in figure (3-5). Here the coordinates become altitude H above the horizon, azimuth Az measured from true north and ρ, distance from the observer.

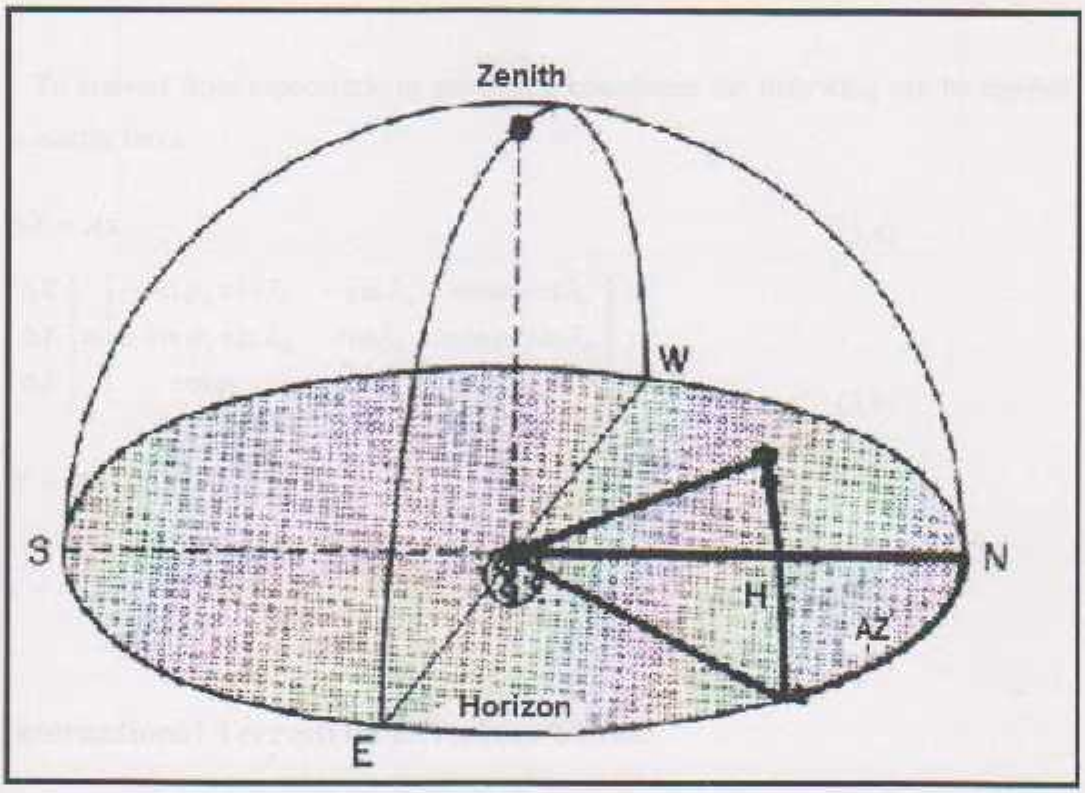


Figure (3-5): Top centric Coordinate System [3]

The position of the point is defined by the zenith (ze), distance (s) and Azimuth (Az) measured clockwise from the north.

Where:

$$
\begin{aligned}
x &= S\cos Az \sin ze \\
y &= S\sin Az \sin ze \qquad (3.6) \\
z &= S\cos ze
\end{aligned}
$$

If geocentric coordinates are used

$$X = \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}, \quad x = \begin{bmatrix} x \\ y \\ z \end{bmatrix} \tag{3.7}$$

To convert from topocentric to geocentric coordinate the following can be applied in matrix form.

$$\Delta X = Ax \tag{3.8}$$

$$\begin{bmatrix} \Delta X \\ \Delta Y \\ \Delta Z \end{bmatrix} = \begin{bmatrix} -\sin\varphi_0 \cos\lambda_0 & -\sin\lambda_0 & \cos\varphi \cos\lambda_0 \\ -\sin\varphi_0 \sin\lambda_0 & \cos\lambda_0 & \cos\varphi_0 \sin\lambda_0 \\ \cos\varphi_0 & 0 & \sin\varphi_0 \end{bmatrix} \begin{bmatrix} x \\ y \\ z \end{bmatrix} \tag{3.9}$$

$$X = X_{p0} + \Delta X$$

$$x = A^{-1}\Delta X = A^{T}\Delta X \tag{3.10}$$

3.4 International Terrestrial Reference Frame

The International Terrestrial Reference Frame is the worldwide adopted standard frame used as basis for all geodetic and geophysical applications. The use of GPS permanent stations for a variety of applications allows in particular the densification and dissemination of ITRF worldwide thanks to IGS high quality products, being reported to ITRF global frame. Point positioning using IGS clock corrections and orbits already expressed in the ITRF permits disseminating the ITRF at the cm-level, without reference to any other point on the Earth surface. The high level of accuracy and consistency of IGS products could not be reached without the revolutionary potential of GPS permanent stations, as shown in figure (3-6).

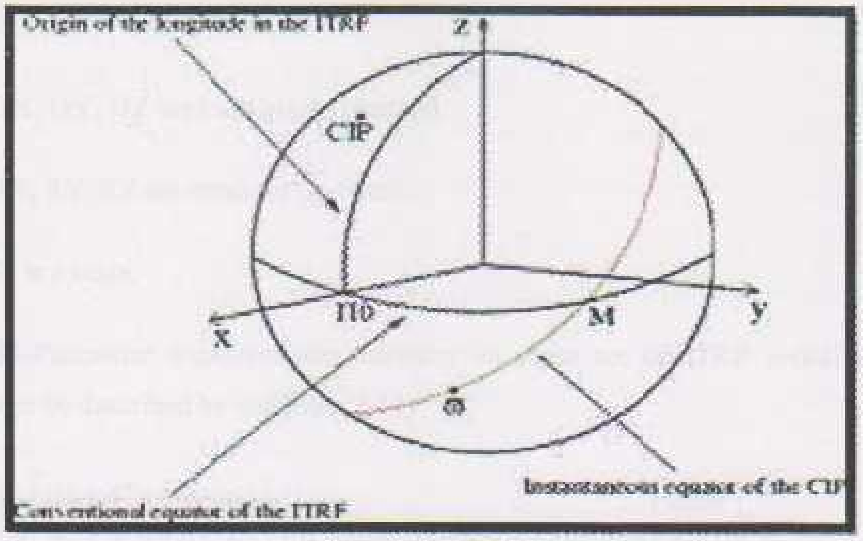


Figure (3-6): ITRF [6]

ITRF coordinates will in general differ from GDA94 coordinates for two main reasons, namely tectonic motion of the Australian land mass and reference frame differences. Tectonic motion of the Australian land mass is approximately 7cm/year in the NNE direction. Differences between the ITRF92 coordinate reference frame and the ITRF2000 are at the several cm in magnitude. A standard 7-parameter transformation can adequately model these differences at the cm level, provided the7-parameter transformation parameters are regularly updated to reflect the tectonic motion. A slightly more complex 14-parameter transformation (7-parameters + their rates) can be used as a better long-term practical solution to these coordinate transformations. A 14 parameter transformation parameter allow users to map a 7 parameter transformation to any epoch of interest.

The 7-Parameter transformation between an input set of ITRF coordinates and GDA94 can be described by equation (3.11).

$$\begin{bmatrix} X_{GDA94} \\ Y_{GDA94} \\ Z_{GDA94} \end{bmatrix} = \begin{bmatrix} dx \\ dy \\ dz \end{bmatrix} + (1 + Sc)* \begin{bmatrix} 1 & r_z & -r_y \\ -r_z & 1 & r_x \\ r_y & -r_x & 1 \end{bmatrix} \begin{bmatrix} X_{ITRF} \\ Y_{ITRF} \\ Z_{ITRF} \end{bmatrix} \qquad (3.11)$$

Where:

X, Y, Z_{GDA94} are the transformed GDA94 Earth centered Cartesian coordinates (meters).

X, Y, Z_{ITRF} are the input ITRF Earth centered Cartesian coordinates (meters).

DX, DY, DZ are translations (meters).

RX, RY, RZ are rotations (radians).

Sc is a scale.

The 14-Parameter transformation between an input set of ITRF coordinates and GDA94 can be described by equation (3.12).

$$\begin{bmatrix} X_{GDA94} \\ Y_{GDA94} \\ Z_{GDA94} \end{bmatrix} = \begin{bmatrix} dx + d^{*}x * (t - t_0) \\ dy + d^{*}y * (t - t_0) \\ dz + d^{*}z * (t - t_0) \end{bmatrix} + \left(1 + Sc + S^{*}c * (t - t_0)\right) * R' * \begin{bmatrix} X_{ITRF} \\ Y_{ITRF} \\ Z_{ITRF} \end{bmatrix} \tag{3.12}$$

$$R' = \begin{bmatrix} 1 & (rz + r^{*}z * (t - t_0)) & -(ry + r^{*} * (t - t_0)) \\ -(rz + r^{*}z * (t - t_0)) & 1 & (rx + r^{*}x * (t - t_0)) \\ (ry + r^{*}y * (t - t_0)) & -(rx + r^{*}x * (t - t_0)) & 1 \end{bmatrix}$$

Where:

X, Y, Z_{GDA94} are the transformed GDA94 Earth centered Cartesian coordinates (meters).

X, Y, Z_{ITRF} are the input ITRF Earth centered Cartesian coordinates (meters).

dx,dy,dz, $d^{*}x$, $d^{*}y$, $d^{*}z$ are translations and their rates (meters, meters/year).

rx,ry,rz, $r^{*}x$, $r^{*}y$, $r^{*}z$ are rotations and their rates (radians, radians/year).

Sc $S^{*}c$ is a scale and its rate (/year).

The parameter t_0 (years) is the reference epoch, and t (years) is the reference epoch of the input ITRF coordinates.

3.5 North Pole Movement

As a first-order approximation, the Earth's magnetic field can be modeled as a simple dipole (like a bar magnet), tilted about 10° with respect to the Earth's rotation axis (which defines the Geographic North and Geographic South Poles) and centered at the Earth's centre. The North and South Geomagnetic Poles are the antipodal points where the axis of this theoretical dipole intersects the Earth's surface. If the Earth's magnetic field were a perfect dipole then the field lines would be vertical at the Geomagnetic Poles, and they would coincide with the Magnetic Poles. However, the approximation is imperfect, and so the Magnetic and Geomagnetic Poles lie some distance apart.

Like the North Magnetic Pole, the North Geomagnetic Pole attracts the north pole of a bar magnet and so is in a physical sense actually a south magnetic pole. It is the centre of the region of the magnetosphere in which the Aurora Borealis can be seen. As of 2005 it was located at approximately 79.74°N 71.78°W, off the northwest coast of Greenland, but it is now drifting away from North America and toward Siberia.

3.6 Tectonic plate movement

Tectonic plate movement, or continental drift as it was commonly referred to, adds another level of complexity to understanding datum. This is because, regardless of the Ellipsoid/Spheroid or reference frame, the various landmasses are moving in relation to each other and in relation to the reference frame.

It is perhaps useful to think of a Ellipsoid/Spheroid and reference frame as establishing a grid over the Earth. Continents move under this grid and therefore the coordinates of any point will change over time, relative to the grid. Australia for instance, moves towards the north-east at approximately 7 centimeters per year.

For accurate work, it is therefore necessary to define a reference time or epoch for positions established using the Ellipsoid/Spheroid and reference frame. Sometimes the velocities of control points are also quoted. For example, assume the corner of a mining lease in Australia was defined using WGS84 coordinates. If the date of the survey is not known and could be anytime over the past 10 years, there is an uncertainty in the location of the corner of 7cm per year or 0.7 of a meter.

CHAPTER FOUR

PRINCIPLES OF LEAST SQUARES

4.1 INTRODUCTION

4.2 FUNDAMENTAL PRINCIPLE OF LEAST SQUARES

4.3 Equal-Weight Case

4.4 LEAST SQUARES SOLUTION OF NONLINEAR SYSTEMS

4.5 LEAST SQUARES ADJUSTMENT OF GNSS NETWORKS

4.6 COORDINATE TRANSFORMATIONS

4.7 HELMERT TRANSFORMATION

4.1 Introduction

In surveying, observations must often satisfy established numerical relationships known as geometric constraints. As examples in a closed polygon traverse, horizontal angle and distance measurements should conform to the geometric constraints, and in a differential leveling loop, the elevation differences should sum to a given quantity However, because the geometric constraints meet perfectly rarely, if ever, the data are adjusted.

Errors in observations conform to the laws of probability; that is, they follow normal distribution theory. Thus, they should be adjusted in a manner that follows these mathematical laws. Whereas the mean has been used extensively throughout history, the earliest works on least squares started in the late eighteenth century. Its earliest application was primarily for adjusting celestial observations. Laplace first investigated the subject and laid its foundation in 1774. The first published article on the subject, entitled method of least squares was written in 1805 by Legendre.

4.2 Fundamental Principle of Least Squares

4.2.1 Fundamental Principale Of Equal Weighted Least

The fundamental principle of a least squares adjustment was developed for observations having equal or unit weights.

$$\sum V^2 = v_1^2 + v_2^2 + \dots\dots\dots + v_n^2 = \min. \tag{4.1}$$

4.2.2 Fundamental Principle Of Weighted Least Squares

The fundamental principle of a least squares adjustment was developed for observations having equal or unit weights. The more general case of least squares adjustment assumes that the observations have varying degrees of precision, and thus varying weights.

$$\sum wV^2 = w_1 v_1^2 + w_2 v_2^2 + \dots\dots\dots + w_n v_n^2 = \min. \tag{4.2}$$

4.2.3 Using Matrices To Form The Normal Equations

Note that the number of normal equations in a parametric least squares adjustment is always equal to the number of unknown variables. Often, the system of normal equations becomes quite large. But even when dealing with three unknowns, their solution by hand is time consuming. As a consequence computers and matrix methods as described in Appendixes A through C are used almost always today. In the following subsections we present the matrix methods used in performing a least squares adjustment.

4.3 Equal-Weight Least Square Solution

To develop the matrix expressions for performing least squares adjustments an analogy will be made with the systematic procedures demonstrated for this development, let a system of observation equations be represented by the matrix notation.

$$AX = L + V \qquad (4.3)$$

Where:

$$A = \begin{bmatrix} a_{11} & a_{12} & \cdots & a_{1n} \\ a_{21} & a_{22} & \cdots & a_{2n} \\ \vdots & \vdots & \vdots & \vdots \\ a_{m1} & a_{m2} & \cdots & a_{mn} \end{bmatrix}$$

$$X = \begin{bmatrix} x_1 \\ x_2 \\ \vdots \\ x_n \end{bmatrix}$$

$$L = \begin{bmatrix} l_1 \\ l_2 \\ \vdots \\ l_n \end{bmatrix}$$

$$V = \begin{bmatrix} v_1 \\ v_2 \\ \vdots \\ v_n \end{bmatrix}$$

4.3.1 Weighted Least Square Solution

$$WAX=WL+WV \tag{4.4}$$

$$W = \begin{bmatrix} w_1 \\ w_2 \\ \vdots \\ w_n \end{bmatrix}$$

4.4 Least Squares Solution Of Nonlinear Systems

Method of solving a nonlinear system of equations using a Taylor series approximation. Following this procedure, the least squares solution for a system of nonlinear equations can be found as follows:

Step 1: Write the first-order Taylor series approximation for each equation.

Step 2: Determine initial approximations for the unknowns in the equations of step 1.

Step 3: Use matrix methods similar to those discussed to find the least squares solution for the equations of step 1.

Step 4: Apply the corrections to the initial approximations

Step 5: Repeat steps 1 through 4 until the corrections become sufficiently small.

A system of nonlinear equations that are linearized by a Taylor series approximation can be written as:

$$JX = K + V \tag{4.5}$$

$$J=\begin{bmatrix} \frac{\delta F_1}{\delta x_1} & \frac{\delta F_1}{\delta x_2} & \cdots & \frac{\delta F_1}{\delta x_n} \\ \frac{\delta F_2}{\delta x_1} & \frac{\delta F_2}{\delta x_2} & \cdots & \frac{\delta F_2}{\delta x_n} \\ \vdots & \vdots & \vdots & \vdots \\ \frac{\delta F_m}{\delta x_1} & \frac{\delta F_m}{\delta x_2} & \cdots & \frac{\delta F_m}{\delta x_n} \end{bmatrix} \qquad X=\begin{bmatrix} dx_1 \\ dx_2 \\ \vdots \\ dx_n \end{bmatrix} \qquad K=\begin{bmatrix} l_1 - f_1(x_1,x_2,\dots,x_n) \\ l_2 - f_2(x_1,x_2,\dots,x_n) \\ \vdots \\ l_m - f_m(x_1,x_2,\dots,x_n) \end{bmatrix} \qquad V=\begin{bmatrix} v_1 \\ v_2 \\ \vdots \\ v_m \end{bmatrix}$$

Where the Jacobean matrix J contains the coefficients of the linearized observation equations.

4.5 Leas Squares Adjustment Of GNSS Networks

Since GNSS networks contain redundant observations, they must be adjusted to make all coordinate differences consistent. In applying least squares to the problem of adjusting baselines in GNSS networks, observation equations are written that relate station coordinates to the coordinate differences observed and their residual errors. For line AB, an observation equation can be written for each baseline component observed as shown in figure(4-1):

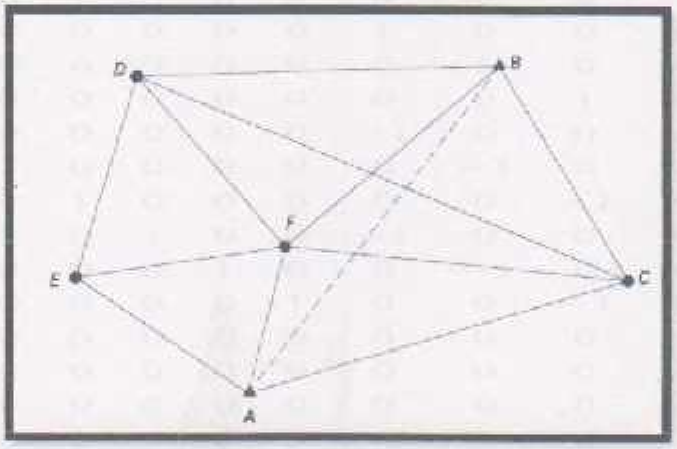


figure (4-1): GNSS network [4]

$$\Delta X_{AB} + v_{x_{AB}} = X_B - X_A \tag{4.6}$$

$$\Delta Y_{AB} + v_{y_{AB}} = Y_B - Y_A \tag{4.7}$$

$$\Delta Z_{AB} + v_{z_{AB}} = Z_B - Z_A \tag{4.8}$$

Similarly, the observation equations for the baseline components of line CD are:

$$\Delta X_{CD} + v_{x_{CD}} = X_D - X_C \tag{4.9}$$

$$\Delta Y_{CD} + v_{y_{CD}} = Y_D - Y_C \tag{4.10}$$

$$\Delta Z_{CD} + v_{z_{CD}} = Z_D - Z_C \tag{4.11}$$

Observation equations of the foregoing form would be written for all measured baselines in . The observation equations can be ex-pressed in matrix form as:

$$AX = L + V \tag{4.12}$$

Where:

$$A = \begin{bmatrix}
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 \\
1 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 & -1 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 & 0 & -1 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & -1 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -1 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -1 \\
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 \\
0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -1 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -1 \\
0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & -1 & 0 & 0 \\
0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & -1 & 0 \\
0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & -1 \\
\vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1
\end{bmatrix}$$

$$V = \begin{bmatrix} v_{X_{AC}} \\ v_{Y_{AC}} \\ v_{Z_{AC}} \\ v_{X_{AE}} \\ v_{Y_{AE}} \\ v_{Z_{AE}} \\ v_{X_{BC}} \\ v_{Y_{BC}} \\ v_{Z_{BC}} \\ v_{X_{BD}} \\ v_{Y_{BD}} \\ v_{Z_{BD}} \\ v_{X_{DC}} \\ v_{Y_{DC}} \\ v_{Z_{DC}} \\ v_{X_{DE}} \\ v_{Y_{DE}} \\ v_{Z_{DE}} \\ v_{X_{FA}} \\ v_{Y_{FA}} \\ v_{Z_{FA}} \\ v_{X_{FC}} \\ v_{Y_{FC}} \\ v_{Z_{FC}} \\ v_{X_{FE}} \\ v_{Y_{FE}} \\ v_{Z_{FE}} \\ \vdots \\ v_{X_{AF}} \\ v_{Y_{AF}} \\ v_{Z_{AF}} \end{bmatrix} \qquad X = \begin{bmatrix} X_C \\ Y_C \\ Z_C \\ X_E \\ Y_E \\ Z_E \\ X_D \\ Y_D \\ Z_D \\ X_F \\ Y_F \\ Z_F \end{bmatrix}$$

The numerical values of the elements of the L matrix are determined by rearranging the observation equations. Its first three elements are for the ΔX, ΔY, and ΔZ baseline components of line AB, respectively. Those elements are calculated as follows:

$$L_X = X_A + \Delta X_{AB} \quad (4.13)$$

$$L_Y = X_A + \Delta X_{AB} \quad (4.14)$$

$$L_Z = X_A + \Delta X_{AB} \quad (4.15)$$

The other elements of the L matrix are formed in the same manner as demonstrated for baseline AC However, before numerical values for the L -matrix elements can be obtained, the Xe , Ye, and Z e geocentric coordinates of all control points in the network must be computed.

Note that the observation equations for GNSS network adjustment are linear and that the only nonzero elements of the A matrix are either 1 or -1. This is the same type of matrix that was developed in adjusting level nets by least squares. In fact, GNSS network adjustments are performed in the very same manner as level net adjustments, with the exception of the weights. In GPS relative positioning, the three observed baseline components are correlated. Therefore, a

Covariance matrix of dimensions 3 * 3 is derived for each base-line as a product of the least squares adjustment of the carrier-phase measurements.

The weight matrix for any GPS network is therefore a block-diagonal type, with an individual 3 * 3 matrix for each baseline observed on the diagonal. When more than two receivers are used, additional 3 * 3 matrices are created in the off-diagonal region of the matrix to provide the correlation that exists between baselines observed simultaneously. All other elements of the matrix are zeros.

4.6 Least Square Solution Of Coordinates Transformation Method

The transformation of points from one coordinate system to another is a common problem encountered in surveying and mapping. For instance, a surveyor who works initially in an assumed coordinate system on a project may find it necessary to transfer the coordinates to the state plane coordinate system. In GNSS surveying and in the field of photogrammetry, coordinate transformations are used extensively. Since the inception of the North American Datum of 1983 (NAD 83), many land surveyors, management agencies state departments of transportation, and others have been struggling with the problem of converting their multitudes of stations defined in the 1927 datum (NAD 27) to the 1983 datum. Although several mathematical models have been developed to make these conversions, all involve some form of coordinate transformation. This chapter covers the introductory procedures of using least squares to compute several well-known and often used transformations. More rigorous procedures, which employ the general least squares procedure.

4.6.1 Coordinat Transformation Two -Dimensionna Conformal

The two-dimensional conformal coordinate transformation, also known as the four-parameter similarity transformation, has the characteristic that true shape is retained after transformation. It is typically used in surveying when converting separate surveys into a common reference coordinate system. This transformation is a three-step process that involves :-

- Scaling to create equal dimensions in the two coordinate systems
- Rotation to make the reference axes of the two systems parallel
- Translation s to create a common origin for the two coordinate systems

The scaling and rotation are each defined by one parameter. The translations involve two parameters. Thus, there are a total of four parameters in this transformation. The transformation requires a minimum of two points called control points, that are common to both systems. With the minimum of two points, the four parameters of the transformation can be determined uniquely. If more than two control points are available, a least squares adjustment is possible.

After determining the values of the transformation parameters, any points in the original system can be transformed.

$$ax_a - by_a + c = X_A + v_{xA}$$

$$ay_a + bx_a + d = Y_A + v_{yA}$$

$$ax_b - by_b + c = X_B + v_{xB}$$

$$ay_b + bx_b + d = Y_B + v_{yB}$$

$$ax_c - by_c + c = X_C + v_{xC}$$

$$ay_c + bx_c + c = Y_c + v_{yC} \tag{4.16}$$

$$AX = L + V \tag{4.17}$$

$$\begin{bmatrix} x_a & -y_a & 1 & 0 \\ y_a & x_a & 0 & 1 \\ x_b & -y_b & 1 & 0 \\ y_b & x_b & 0 & 1 \\ x_c & -y_c & 1 & 0 \\ y_c & x_c & 0 & 1 \end{bmatrix} \begin{bmatrix} a \\ b \\ c \\ d \end{bmatrix} = \begin{bmatrix} X_A \\ Y_A \\ X_B \\ Y_B \\ X_C \\ Y_C \end{bmatrix} + \begin{bmatrix} v_{XA} \\ v_{YA} \\ v_{XB} \\ v_{YB} \\ v_{XC} \\ v_{YC} \end{bmatrix}$$

4.6.2 Two- Dimensional Affine Coordinat Transformation

The two-dimensional affine coordinate transformation, also known as the six parameter transformation, is a slight variation from the two-dimensional conformal transformation. In the affine transformation there is the additional allowance for two different scale factors; one in the x direction and the other in the y direction. This transformation is commonly used in photogrammetry for interior orientation. That is, it is used to transform photo coordinates from an arbitrary measurement photo coordinate system to a camera fiducial system and to account for differential shrinkages that occur in the x and y directions. As in the conformal transformation,the affine transformation also applies two translations of the origin, and a rotation about the origin, plus a small non- orthogonality correction between the x and y axes. This results in a total of six unknowns. The mathematical model for the affine transformation is:

$$ax + by + c = X + V_x \qquad (4.18)$$

$$dx + ey + f = Y + V_y \qquad (4.19)$$

These equations are linear and can be solved uniquely when three control points exist (i.e., points whose coordinates are known in the both systems).

This is because for each point, an equation set in the form of Equations (4.21) can be written, and three points yield six equations involving six unknowns.

If more than three control points are available, a least squares solution can be obtained. Assume, for example, that four common points (1, 2, 3, and 4)

exist. Then the equation system would be:

$$\begin{aligned}
&ax_1 + by_1 + c = X_1 + V_{x1}\\
&dx_1 + ey_1 + f = Y_1 + V_{y1}\\
&ax_2 + by_2 + c = X_2 + V_{x2}\\
&dx_2 + ey_2 + f = Y_2 + V_{y2}\\
&ax_3 + by_3 + c = X_3 + V_{x3}\\
&dx_3 + ey_3 + f = Y_3 + V_{y3}\\
&ax_4 + by_4 + c = X_4 + V_{x4}\\
&dx_4 + ey_4 + c = Y_4 + V_{y4} \qquad (4.20)
\end{aligned}$$

In matrix notation, Equations (4.21) are expressed as $AX = L + V$, where:

$$\begin{bmatrix} x_1 & y_1 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & x_1 & y_1 & 1 \\ x_2 & y_2 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & x_2 & y_2 & 1 \\ x_3 & y_3 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & x_3 & y_3 & 1 \\ x_4 & y_4 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & x_4 & y_4 & 1 \end{bmatrix} \begin{bmatrix} a \\ b \\ c \\ d \\ e \\ f \end{bmatrix} = \begin{bmatrix} X_1 \\ Y_1 \\ X_2 \\ Y_2 \\ X_3 \\ Y_3 \\ X_4 \\ Y_4 \end{bmatrix} + \begin{bmatrix} v_{x1} \\ v_{y1} \\ v_{x2} \\ v_{y2} \\ v_{x3} \\ v_{y3} \\ v_{x4} \\ v_{y4} \end{bmatrix} \qquad (4.21)$$

4.6.3 Two - Dimensional Projective Coordinat Transformation

The two-dimensional projective coordinate transformation is also known as the eight-parameter transformation. It is appropriate to use when one two dimensional coordinate system is projected onto another nonparallel system This transformation is commonly used in photogrammetry and it can also be used to transform NAD 27 coordinates into the NAD 83 system. In their final form, the two-dimensional projective coordinate transformation equations are:

$$X = \frac{a_1 x + b_1 y + c}{a_3 x + b_3 + 1} \qquad (4.22)$$

$$Y = \frac{a_2 x + b_2 y + c}{a_3 x + b_3 + 1} \qquad (4.23)$$

Upon inspection, it can be seen that these equations are similar to the affine transformation. In fact if and were equal to zero these equations are the affine transformation. With eight unknowns, this transformation requires a minimum of four control points. If there are more than four control points, the least squares solution can be used. Since these are nonlinear equations, they must be linearzed the linearzed form of these of these equations:

$$\begin{bmatrix} \left(\frac{\delta X}{\delta a_1}\right)_0 & \left(\frac{\delta X}{\delta b_1}\right)_0 & \left(\frac{\delta X}{\delta c_1}\right)_0 & 0 & 0 & 0 & \left(\frac{\delta X}{\delta a_3}\right)_0 & \left(\frac{\delta X}{\delta b_3}\right)_0 \\ 0 & 0 & 0 & \left(\frac{\delta X}{\delta a_2}\right)_0 & \left(\frac{\delta X}{\delta b_2}\right)_0 & \left(\frac{\delta X}{\delta c_2}\right)_0 & \left(\frac{\delta X}{\delta a_3}\right)_0 & \left(\frac{\delta X}{\delta b_3}\right)_0 \end{bmatrix} \begin{bmatrix} da_1 \\ db_1 \\ dc_1 \\ da_2 \\ db_2 \\ dc_2 \\ da_3 \\ db_3 \end{bmatrix} =$$

$$\begin{bmatrix} X - X_0 \\ Y - Y_0 \end{bmatrix} \qquad (4.24)$$

4.6.4 Three -Dimensional Conformal Coordinat Transformation

The three-dimensional conformal coordinate transformation is also known as the seven-parameter similarity transformation . It transfers points from one three-dimensional coordinate system to another. It is applied in the process of reducing data from GPS surveys and is also used extensively in photo grammetry. The three-dimensional conformal coordinate transformation in solves seven parameters, three rotations, three translations, and one scale factor. The rotation matrix is developed from three consecutive two dimensional rotations about the x, y, and z axes, respectively. Given in sequence, these are as follows:

$$X = S(r_{11}x + r_{21}y + r_{31}z) + T_x \qquad (4.25)$$

$$Y = S(r_{12}x + r_{22}y + r_{32}z) + T_y \qquad (4.26)$$

$$Z = S(r_{13}x + r_{23}y + r_{33}z) + T_z \qquad (4.27)$$

For a unique solution, seven equations must be written. This requires a minimum of two control stations with known XY coordinates and also xy coordinates, plus three stations with known Z and z coordinates. If there is more than the minimum number of control points, a least-squares solution can be used.

$$\begin{bmatrix} \left(\frac{\delta X}{\delta S}\right)_0 & 0 & \left(\frac{\delta X}{\delta\theta_2}\right)_0 & \left(\frac{\delta X}{\delta\theta_3}\right) & 1 & 0 & 0 \\ \left(\frac{\delta Y}{\delta S}\right)_0 & \left(\frac{\delta Y}{\delta\theta_1}\right)_0 & \left(\frac{\delta Y}{\delta\theta_2}\right)_0 & \left(\frac{\delta Y}{\delta\theta_3}\right) & 0 & 1 & 0 \\ \left(\frac{\delta Z}{\delta S}\right)_0 & \left(\frac{\delta Z}{\delta\theta_1}\right)_0 & \left(\frac{\delta Z}{\delta\theta_2}\right)_0 & \left(\frac{\delta Z}{\delta\theta_3}\right)_0 & 0 & 0 & 1 \end{bmatrix} \begin{bmatrix} dS \\ d\theta_1 \\ d\theta_2 \\ d\theta 3 \\ dT_x \\ dT_y \\ dT_z \end{bmatrix} = \begin{bmatrix} X - X_0 \\ Y - Y_0 \\ Z - Z_0 \end{bmatrix} \qquad (4.28)$$

4.7 Helmert Transformation

Local datum's such as NAD 83 are known as Earth-centered, Earth-axed (ECEF) coordinate systems. This means that the Z axis is nearly aligned with the Conventional Terrestrial Pole, X axis with the Greenwich Meridian, and the origin is at the mass center of the Earth as derived by the datum points used in the dentition. International datum's such as the International Terrestrial Reference Frame use the same dentitions for the axes, origin, and ellipsoid, but differ slightly due to the difference in the datum points used in its determination. Thus, the rotational parameters and translations between two ECEF coordinate systems are usually very small. The scale factor between two datum's using the same units of measure should be nearly 1.

$$R = \begin{bmatrix} 1 & \theta_3 & -\theta_2 \\ -\theta_3 & 1 & \theta_1 \\ \theta_2 & -\theta_1 & 1 \end{bmatrix} = \mathrm{I} + \begin{bmatrix} 0 & \Delta\theta_3 & -\Delta\theta_2 \\ -\mathrm{X}\theta_3 & 0 & \Delta\theta_1 \\ \Delta\theta_2 & -\Delta\theta_1 & 0 \end{bmatrix} = \mathrm{I} + \Delta R \qquad (4.29)$$

The transformation of coordinates from one local datum to another datum is performed as:

$$X_{LD} = sRX_{GD} + T \qquad (4.30)$$

CHAPTER FIVE

FIELDWORK

5.1 Introduction

As mentioned in the project timeline, a field work is to performed in order to get a precise GNSS point readings, this work is divided into many stages; exploration of points on the area specified is to be done before the observation of points, also these points are located on some areas of personal property or it may be doesn't existing due to the early time of 1932 when the point where installed.

5.2 Getting familiar into using GNSS devices

Using GNSS based devices is roughly known for any researcher or a student of survey, hence, the use of GNSS static method that was described before is to be practiced in order to master the task needed to get the necessary data.

The initial stages of GNSS practical training on the devices is performed uing two brands of GNSS apparatus which are:

1. Trimble R8
2. Trimble 5700

Ultimately, the main structure of the devices is familiar, also using them became familiar and team is ready for the stage of point observations, as shown in figure (5-1).

Figure (5-1): R8 GNSS components

5.3 Selection of Points

Points that are needed is ones of known and briefly precise 3-dimensional coordinates, these are the trigonometric points fixed during the British mandate on Palestine.

The actual preparations for setting up a triangulation system commenced only in February 1921 after the Survey Department moved to its new home in Jaffa; the survey began in May 1921The first step was for the survey parties to lay out geodetic points throughout the entire country, to measure their values, and to provide mathematical bases for the survey nets. The geodetic points required for mapping are classed in three categories:

1) Fixed points, or trigonometric stations, are determined by trigonometric methods and must be in sight of each other for the surveying observations. These imaginary lines form the sides of the triangles of the observation net. The data obtained are the position of the points in planimetric coordinates. The elevation of the points is determined in relation to the datum (reference) level, which is the mean sea level (MSL).

2) Spot heights are determined by precise levelling and not necessarily in relation to the trigonometric net. The topographic heights are measured in relation to the MSL along fixed runs in the field.

3) Gravimetric points, for the determination of the figure of the Earth.

As the points of trigonometric stations are mostly available with their names and position values, so the points of project network is chosen from these points, a suitable coverage of an area within Hebron district is chosen according to the availability of points and ease of accessibility to them. Figure (5.2) illustrates the distribution of these points within the area of study.

Table (5.1) also shows the information about points' names and coordinates also it provides the location of the point according to the nearest town or city.

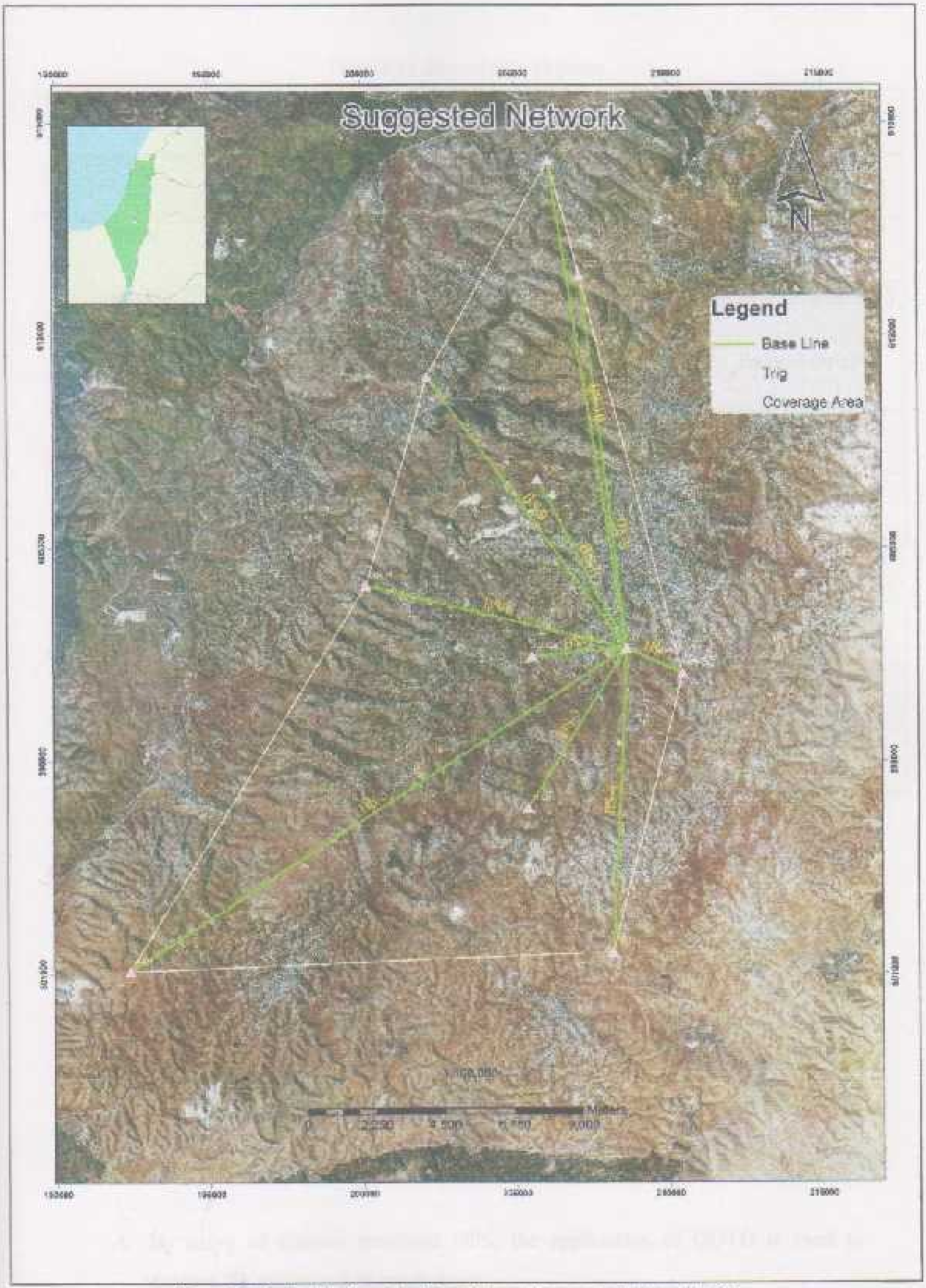


Figure (5-2): distribution of points within study area in Hebron district

Table (5-1): Selected network points

Point(name)	Easting(m)	Northing(m)	Elevation(m)	Location
552B	155580.17	101424.37	913.81	Dura (Sinjer)
356N	155722.87	107271.25	875.47	Hebron (Looza)
441F	142397.9	91081.11	643.29	Aldahriyeh(Ramadeen)
520T	157133.47	113959.94	849.42	Halhoul (Jala)
351N	150135.28	103756.06	730.17	Dura(Homsa)
597B	160474.73	100867.46	902.79	Hebron (Qilqis mt.)
373N	155409.64	96442.86	739.5	Yatta (beit Emra)
389N	158184.65	91685.62	752.37	Yatta (Sosya)
347N	152144.28	110606.8	567.75	Beit Oula
570T	156096.76	117739.33	588.94	Sourif (town center)

5.4 Methodology of point's selection

1. List of points and their coordinates are obtained from the Palestinian department of land and survey.
2. Benefiting from an orthophoto of the West Bank, and by conversion of the txt file of trigs into a shapefile in ArcMap. The location of points can be located.
3. The availability of points are not determined by a theoretical database, so exploration of the points is the only way to get into points.
4. By using of Garmin handheld GPS, the application of GOTO is used to explore the points and to reach them.

5. A resulting set of 10 points are available and explored, so they are just ready to be observed.

5.5 List of network points

Each point of the network is described either by coordinates and aerial photos are listed as follows:

1. Point 522B: this point is located at the hill of Sinjer neighborhood in Dura, it's located on a rock at the uphill. Figure (5-3) shows an orthophoto for the point mentioned

Figure (5-3): Point 522B

2. Point 356N: this point is located at the hill in looza neighborhood in Dura, it's located on a rock at the uphill under Oak tree. Figure (5-4) shows an orthophoto for the point mentioned

Figure (5-4): Point 356N

3. Point 356N: this point is located on beat-Oula, it's located between two properties. Figure (5-5) shows an orthophoto for the point mentioned

Figure (5-5): Point 347N

4. Point 351N: this point is located on Dura and it's located on a rock on uphill of Homsa neighborhood. Figure (5-6) shows an orthophoto for the point

Figure (5-6): Point 351N

5. Point 389N: this point is located on Yatta and it's located on a rock on uphill of Sosya neighborhood. Figure (5-7) shows an orthophoto for the point.

Figure (5-7):Point 389N

6. Point 441F: this point is located on Aldahreyeh and it's located on a rock on uphill near Alramadeen neighborhood. Figure (5-8) shows an orthophoto for the point mentioned.

Figure (5-8): point 441F

7. Point 373N: this point is located on Yatta and it's located on a rock on Beietemra Figure (5-9) shows an orthophoto for the point mentioned

Figure (5-9): Point 373N

8. Point 520T: this point is located on Halhul near Jala street, Figure (5-10) shows an orthophoto for the point mentioned.

Figure (5-10): point 520T

9. Point 570T: this point is located on center of Sourif town on the center of a cross vault of a house near the main mosque, Figure (5-11) shows an orthophoto for the point.

Figure (5-11): point 570T

10. Point 597b: this point is located on Hebron city at the top of Qilqis mount south to City center, Figure (5.12) shows an orthophoto for the point mentioned.

Figure (5-12): point 597B

5.6 Mission planning

In planning to go for using GNSS observation, it's important to choose a suitable time that provides a good results and precise work. Since GNSS depends on the constellation of the satellites that affects the DOP, consequently, that affects the quality and accuracy of observation got, it's important to be in the suitable time and place to measure points.

Many of tasks should be accomplished before heading with apparatus to the field, and planning for GNSS/GPS working session should be performed before getting involved in the field, this can be performed following several useful and important tips

1. Firstly, the scope of the region to be measured should be determined, in order to decide which points can be reached and in same case that satellite signals can reach it without interruption or loss due the nature of the region (may be congested area, high rise buildings around etc.).
2. Checking for the satellite constellation during surveying, since that will highly affect the measurements. Recently, GNSS/GPS receivers manufacturers distribute software that provide information about the available visible satellites at specific points to be measured, it's presented as a sky plot for the area to be worked on with the available satellites and values of DOPs that can

make it clear for the surveyor to determine the best time of best constellation, more satellites and least DOP

3. Checking for the weather forecast just before going to field so as to decide whether is it appropriate to work under certain circumstances.

5.7 Data Collection

Heading to field to get data from GNSS observation needs an arrangement for data, RINEX files that are stored in the memory of the receiver is used, also an arrangement for the data observed is to be documented using a data collection sheet as follows on the next page. Table (5-2)

Table(5-2):GNSS Point Measurement Sheet

Point ID.______________

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.	**Date: / / 2013**	
Instrument:	**Output File:**	
GNSS Type: static	**Δ Base:**	**Δ Rover:**
Antenna Height		
Measurement Start Time		
Measurement Finish Time		
Description of location		
Palestinian Coordinates system (ITRF)	**Point shot**	

CHAPTER SIX

CALCULATIONS AND RESULTS

6.1 INTRODUCTION

6.2 AUTO GIPSY.

6.3 ITRF 2008 COORDINATES .

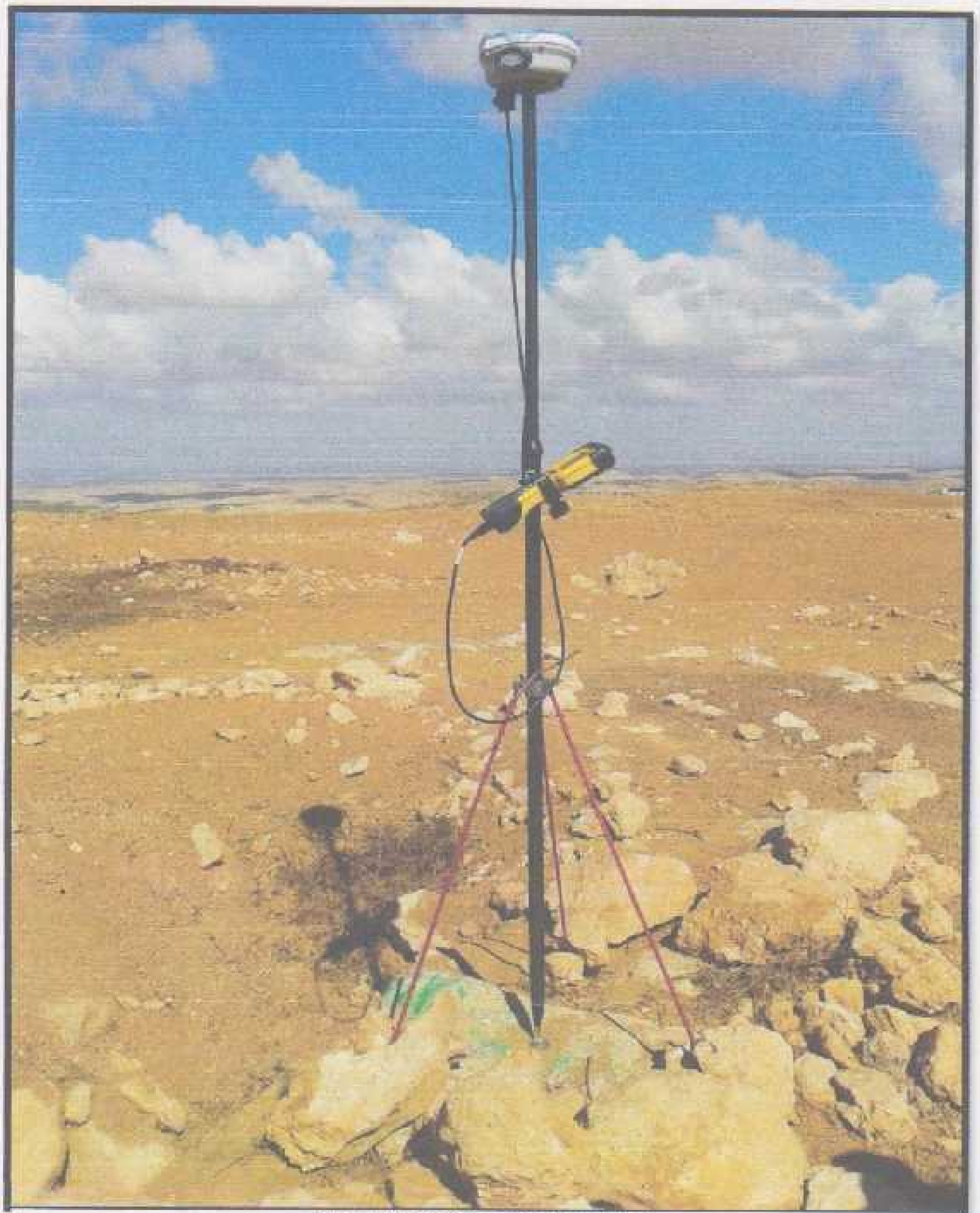
6.4 GEOGRAPHIC COORDINATES.

6.5 PALESTINIAN COORDINATES SYSTEM.

6.6 TRANSFORMATION PARAMETERS.

6.1 Introduction:

After we have reconnaissance the points (Trigs)at the previous semester we have observed each point by using GPS instrument for more than one hour figure (6.1) shows an observed point. After we have finished the observations of all points we have convert the GPS files to rinex file by using the program (Convert To Rinex).Then we upload this files to Auto Gipsy website, this website give us the



Figure(6.1):observed point (441F)

ITRF(2008) coordinates of the points. After that we transform the local Palestinian coordinates to projected Palestinian coordinates by using Geo Transformation program. Then by applying helmert transformation we find the parameters of transformation in order to connect the Palestinian coordinates to ITRF(2008) coordinates .

6.2 Auto Gipsy :

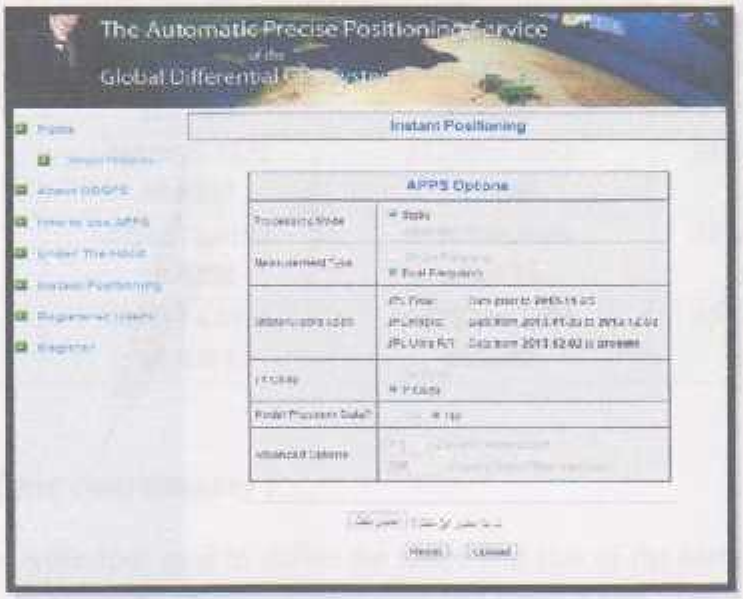
APPS accepts GPS measurement files, and applies the most advanced GPS positioning technology from NASA's Jet Propulsion Laboratory to estimate the position of your GPS receivers, whether they are static, in motion, on the ground, or in the air. APPS employs:

- Real-time GPS orbit and clock products from JPL's GDGPS System.
- JPL's daily and weekly precise GPS orbit and clock products.
- JPL's GIPSY-OASIS software for processing the GPS measurements.

APPS continues to provide JPL's venerable Auto GIPSY (AG) service – for free , for static post-processing (e.g. measurement latency of a week or more), but also offers new and unique services:

- Real-time GPS orbit and clock products from JPL's GDGPS System.
- JPL's daily and weekly precise GPS orbit and clock products.
- JPL's GIPSY-OASIS software for processing the GPS measurements.

Figure(6.2) shows how to upload an rinex file to Auto Gipsy:



Figure(6.2):Auto Gipsy website

6.3 ITRF 2008 coordinate:

ITRF 2008 is the new realization of the International Terrestrial Reference System. Following the procedure already used for the ITRF2005 formation, the ITRF2008 uses as input data time series of station positions and Earth Orientation Parameters (EOPs) provided by the Technique Centers of the four space geodetic techniques (GPS, VLBI, SLR, DORIS). Based on completely reprocessed solutions of the four techniques, the ITRF2008 is expected to be an improved solution compared to ITF2005.Table(6.1) shows the ITRF 2008 coordinates of the points according to Auto Gipsy calculation service (http: apps.gdps.net/apps_file_apload.php)

Table (6.1) ITRF 2008 coordinates of points according to auto gipsy:

Point(#)	E(m) ±σE(m)	N(m) ±σN(m)	Z(m) ±σh(m)
570T	4448567.6855 ±0.0266	3122284.7445 ± 0.0189	3327965.9809 ±0.0132
441F	4467864.7195 ±0.0370	3119132.4383 ±0.0140	3305246.2301 ±0.0141
520T	4449772.1203 ±0.0503	3124402.3732 0.0247	3324884.9216 ±0.0171
597B	4453486.0860 ±0.0199	3131112.5458 ±0.0242	3313757.7139 ±0.0159
347N	4453880.2913 ±0.1549	3121195.1887 ±0.0758	3321874.9661 ±0.1714
356N	4453464.8009 ±0.0399	3125281.5228 ±0.0598	3319198.6618 ±0.0289
351N	4458077.1532 ±0.0287	3121696.6952 ±0.0269	3316118.2539 ±0.0100
552B	4456072.4103 ±0.0258	3126946.3606 ±0.0179	3314234.2880 ±0.0126
373N	4458174.2466 ±0.0374	3128221.1284 ±0.0392	3309893.9461 ±0.0137

6.4 Geographic coordinates :

A geodetic datum is the tool used to define the shape and size of the earth, as well as the reference point for the various coordinate systems used in mapping the earth. Throughout time, hundreds of different datums have been used - each one changing with the earth views of the times. Within the World Geodetic System(WGS), there are several different datums that have been in use throughout the years. These are WGS 84, 72, 70, and 60. Auto Gipsy website used geodetic system called GRS80 with the following parameters that are listed in table (6.2):

Table(6.2):parameters of (GRS80)

Parameter	Value(m)
Semi major axis(a)	6378137
Semi minor axis(b)	6356752.3141
Flattening (f)	1:298.257222101
First excentricity(e^2)	0.00669438002290

The Geographic coordinates (GRS80) of points according to Auto Gipsy of the points are listed in table(6.3)

Table (6.3): Geographic coordinates (GRS80) of points according to Auto Gipsy:

Point(#)	Lon(deg) ±□ Lon(m)	Lat(deg) ±б Lat(m)	H(m) ±бh(m)
570T	35.06359326 ±0.0126	31.65200666 ± 0.0088	609.6782 ±0.0317
441F	34.91989644 ±0.0180	31.411341690 ±0.0099	663.4462 ±0.0366
520T	35.07456802 ±0.0401	31.6179363341 0.0197	868.8533 ±0.0378
597B	35.10990263 ±0.0227	31.4998848000 ±0.0114	921.6776 ±0.0243
347N	35.02204579 ±0.0898	31.5876263800 ±0.0933	587.4169 ±0.2058
356N	35.05979802 ±0.0633	31.5575931800 ±0.0148	895.2368 ±0.0421
351N	35.00101277 ±0.0238	31.52580931 ±0.0162	748..9729 ±0.0287
552B	35.05837632 ±0.0132	31.504860020 ±0.0084	933.5242 ±0.0300
373N	35.05665236 ±0.0344	31.45993032 ±0.0223	758.4460 ±0.0381

6.5 Palestinian Coordinate system (Palestinian 1923 Grid) :

By using Helmert method that was explained in chapter four section (4.7) to transform the local Palestinian coordinates to projected Palestinian coordinates and we have the following results at table (6.4).

Table(6.4): Palestinian Projected Coordinate of points :

Point #	E(m)	N(m)	Z(m)
570T	4448791.5674	3122353.4766	3327694.2194
441F	4468089.1457	3119203.9746	3304974.8240
520T	4449996.5640	3124472.2335	3324613.8102
597B	4453710.0587	3131183.4367	3313487.2212
347N	4454104.5263	3121265.0174	3321603.6519
356N	4453688.9070	3125351.2501	3318927.4228
351N	4458301.7383	3121767.3886	3315847.2546
552B	4456296.2662	3127016.5720	3313962.9819
373N	4458398.4128	3128291.8818	3309622.9585

By using the Helmert method that was explained in chapter four section (4.7) to transform the local Palestinian coordinates to Geodetic coordinates (clarck 1880) Palestinian coordinates and we have the following results at table (6.5).

Table (6.5) Geodetic coordinates (clarck 1880)of points

Point #	Lon(deg)	Lat(deg)	H(m)
570T	35.062830943	31.651679121	588.90
441F	34.919159997	31.410995243	643.29
520T	35.073811069	31.617604927	849.42
597B	35.109153773	31.499554884	902.79
347N	35.021292509	31.587294971	567.75
356N	35.059043599	31.557261929	875.47
351N	35.000266492	31.525473928	730.17
552B	35.057627477	31.504525988	913.81
373N	35.055907088	31.459594269	739.50

6.6 Transformation Parameters :

After applying helmert transformation equations we have the following seven parameters in that will be used to connect the Palestinian coordinates to ITRF(2008) coordinates .

The seven parameters are:

1. Scale.
2. Rotation angle around the X axis (ω).
3. Rotation angle around the Y axis (ϕ).
4. Rotation angle around the X axis (k).
5. Translation of X axis (Tx).
6. Translation of Y axis (Ty).
7. Translation of Z axis (Tz).

6.6.1 First Iteration:

The first solution include all points applied to helmert equations and the parameters values are listed in table (6.6):

Table (6.6): parameters of transformation:

Parameter	value
S	1.000021895
ω(sec)	-12.01661327
ϕ(sec)	-9.695480207
k(sec)	-6.903782457
Tx(m)	-92.12292480
Ty(m)	-71.59912109
Tz(m)	-76.71679688

after finding the transformation parameters we calculate the residuals of the points , table(6.7) shows the residuals of points.

Table(6.7): residuals of points

Point #	Vx(cm)	Vy(cm)	Vz(cm)
570T	4.35	3.61	-15.23
441F	13.62	-33.71	13.68
552B	-16.93	5.33	8.38
597B	-23.54	19.26	9.19
520T	7.50	-13.34	-3.24
347N	25.37	-18.42	-21.20
351N	26.55	-29.60	-11.14
373N	-26.42	11.89	21.31
356N	-11.03	54.90	-2.02

6.6.2 Final Iteration :

According to the values of the residuals at first solution there are some points that have values greater than 30 cm(570T+356N) we make an elimination of this points and we applying again helmret method in order to find the final transformation parameters. Table(6.8) shows the final transformation parameters.

Table(6.8): final transformation parameters:

Parameter	Value
S	1.00001875578891
ω(sec)	-11.2608595134273
ɸ(sec)	-9.3074520912301
k(sec)	-6.06351711094535
Tx(m)	-84.6326293945312
Ty(m)	-55.69677734375
Tz(m)	-63.2705078125

After we calculate the new the final values of residuals after eliminating the points with high value of residuals. Table(6.9) shows the finale values of residuals.

Table(6.9):final values of residuals:

Point #	Vx(cm)	Vy(cm)	Vz(cm)
441F	8.90	-31.84	18.45
552B	-16.42	12.92	5.31
597B	-20.44	26.42	4.25
520T	6.98	1.49	-9.93
347N	22.75	-5.42	-25.06
351N	23.92	-20.53	-12.54
373N	-25.26	16.59	19.48

CHAPTER SEVEN

CONCLUSION

7.1 Introduction.

7.2 Conclusion

7.1Introduction

After finishing all the calculation; we have analyzed the results we have got from the calculations. We have to eliminated some points from the network and we have to select a new points according to their locations.

7.2 conclusion

After the analysis of the results from calculations, we have concluded the following results:

1. We developed a network of the seven calculation points that was distributed around Hebron district.
2. We have accepted points with residual less than 30cm
3. We have observed nine points but we have element two points because their residua, were greater than 30 cm and those points are .
    - 570T
    - 356N
4. We have accepted a point with residual greater than 30 cm in order to cover more area, in the southern part of the district.
5. Finally, we have made the transformation between ITRF2008 system and Palestinian coordinate system(Pal 1923 Grid) of the net work in Hebron district as shown in figure(7.1).

Figure(7.1):final network

Appendix A

A-1 INTRODUCTION

A-2 OBSERVED POINTS

A-1 Introduction.

This appendix shows the point that we have observed in order to develop the final network.

A-2 observed points.

The following table shows the point that we have observed in our project to develop the net work as shown in table (A-1).

Table (A-1): observed points.

Point(name)	Easting(m)	Northing(m)	Elevation(m)	Location
552B	155580.17	101424.37	913.81	Dura (Sinjer)
356N	155722.87	107271.25	875.47	Hebron (Looza)
441F	142397.9	91081.11	643.29	Aldahriyeh(Ramadeen)
520T	157133.47	113959.94	849.42	Halhoul (Jala)
351N	150135.28	103756.06	730.17	Dura(Homsa)
597B	160474.73	100867.46	902.79	Hebron (Qilqis mt.)
373N	155409.64	96442.86	739.5	Yatta (beit Emra)
347N	152144.28	110606.8	567.75	Beit Oula
570T	156096.76	117739.33	588.94	Sourif (town center)

- **Point ID. 570T**

This point was above an mountain and its basic information of points and the information of the rover and base are installed in table(A-1):

Table(A-2): point 570T details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.	**Date:** 22 / 9 / 2013	
Instrument: Trimble 5700	**Output File: Rinex 2.11 file**	
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	**2 m**	
Measurement Start Time	**9:30 am**	
Measurement Finish Time	**11:00 am**	
Description of location	**Above an old building**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4448791.5674 m **Y= 3122353.4766 m** **Z= 3327694.2194 m**		

- **Point ID. 441F**

This point was above an mountain and its basic information of points and the information of of the rover and base are installed in table(2):

Table(A-3): point 441F details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.	**Date: 3 / 10 / 2013**	
Instrument: Trimble 5700	**Output File: Rinex 2.11 file**	
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	2 m	
Measurement Start Time	**9:30 am**	
Measurement Finish Time	**11:00 am**	
Description of location	**Above an amounting**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4468089.1457 m **Y= 3119203.9746 m** **Z= 3304974.8239 m**		

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- **Point ID. 520T**

This point was above an mountain and its basic information of points and the information of of the rover and base are installed in table(3):

Table(A-4): point 520T details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.		**Date:** 10 / 10 / 2013
Instrument: Trimble 5700		**Output File: Rinex 2.11 file**
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	**2 m**	
Measurement Start Time	**11:00 am**	
Measurement Finish Time	**1:00 pm**	
Description of location	**Above an amounting**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4449996.5640 m **Y= 3124472.2335 m** **Z= 3324613.8102 m**		

- **Point ID. 597B**

This point was above an mountain and its basic information of points and the information of of the rover and base are installed in table(4):

Table(A-5): point 597B details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.	Date: 10 / 10 / 2013	
Instrument: Trimble 5700	**Output File: rinex 2.11 file**	
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	2 m	
Measurement Start Time	11:00 am	
Measurement Finish Time	1:00 pm	
Description of location	**Above an amounting**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4453710.0587 m Y= 3131183.4367 m Z= 3313487.2212 m		

- **Point ID. 347N**

This point was above an mountain and its basic information of points and the information of of the rover and base are installed in table(5):

Table(A-6): point 347N details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.	**Date:** 15 / 10 / 2013	
Instrument: Trimble 5700	**Output File: rinex 2.11 file**	
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	**2 m**	
Measurement Start Time	**10:00 am**	
Measurement Finish Time	**11:30 am**	
Description of location	**Above an amounting**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4454104.5263 m **Y= 3121265.0174 m** **Z= 3321603.6519 m**		

- **Point ID, 356N**

This point was above an mountain and its basic information of points and the information of of the rover and base are installed in table(6):

Table(A-7): point 356N details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.	**Date:** 20 / 10 / 2013	
Instrument: Trimble 5700	**Output File: rinex 2.11 file**	
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	**2 m**	
Measurement Start Time	**9:30** am	
Measurement Finish Time	**11:30 pm**	
Description of location	**Above an amounting**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4453688.9070 m **Y= 3125351.2501 m** **Z= 3318927.4228 m**		

- **Point ID. 351N**

This point was above an mountain and its basic information of points and the information of of the rover and base are installed in table(7):

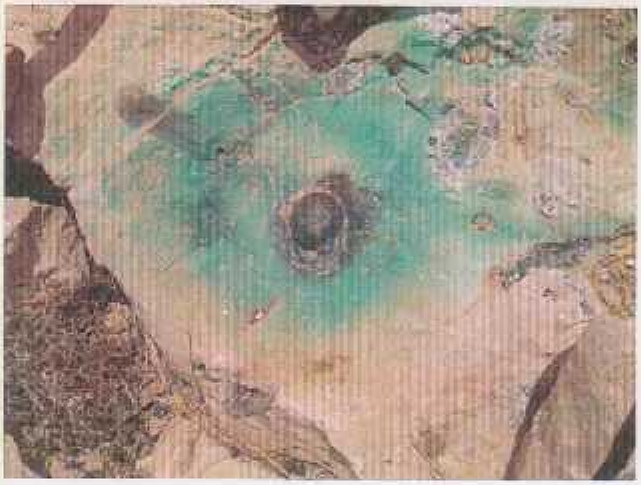
Table(A-8): point 351N details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.	**Date:** 27 / 10 / 2013	
Instrument: Trimble 5700	**Output File: rinex 2.11 file**	
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	**2 m**	
Measurement Start Time	**10:00 am**	
Measurement Finish Time	**11:30 am**	
Description of location	**Above an amounting**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4454104.5263 m Y= 3121265.0174 m Z= 3318927.4227 m		

- **Point ID. 552B**

This point was above an mountain and its basic information of points and the information of of the rover and base are installed in table(8):

Table(A-9): point 552B details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidl.	**Date:** 5 / 11 / 2013	
Instrument: Trimble 5700	**Output File: rinex 2.11 file**	
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	**2 m**	
Measurement Start Time	**10:00 am**	
Measurement Finish Time	**11:30 am**	
Description of location	**Above an amounting**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4456296.2661m **Y= 3127016.5720 m** **Z= 3313962.9818 m**		

- **Point ID. 373N**

This point was above an mountain and its basic information of points and the information of of the rover and base are installed in table(9):

Table(A-10): point 373N details

Team: Abu-zneed ,Sabateen, Abu-latifa, Al-Jaidi.	**Date:** 12 / 11 / 2013	
Instrument: Trimble 5700	**Output File: rinex 2.11 file**	
GNSS Type: static	**Δ Base: Trimble 5700**	**Δ Rover: Trimble 5700**
Antenna Height	**2 m**	
Measurement Start Time	**10:00 am**	
Measurement Finish Time	**11:30 am**	
Description of location	**Above an amounting**	
Palestinian Coordinates system (ITRF)	**Point shot**	
X= 4458398.4128 m **Y= 3128291.8818 m** **Z= 3309622.9585 m**		

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