Palestine Polytechnic University

College of Engineering

Civil and Architecture Engineering Department

Introduction / Graduation Project

Review and Update of the Palestinian GNSS Network (1999)

Project Team :

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Project Supervisor:

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Palestine Polytechnic University

Hebron – Palestine

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In accordance with the recommendation of project supervisor and acceptance of all examining committee members, this project has been submitted to the Department of Civil and Architectural Engineering in the college of Engineering and Technology in partial fulfillment of requirements of the department for degree of Bachelor of Surveying and Geomatics Engineering**.**

Signature of Project Supervisor | **Signature of Department Chairman**

Name……………………………
Name ……………………………

Hebron – Palestine

2017

بسم الله الرحمن الرحيم

قَالُواْ سُبْحَٰنَكَ لَا عِلْمَ لَنَآ إِلَّا مَا عَلَّمْتَنَآ
إِنَّكَ أَنتَ ٱلْعَلِيمُ ٱلْحَكِيمُ

صدق الله العظيم

(32 سورة البقرة)

I dedicate my dissertation work to my family and many friends. A special feeling of gratitude to my loving parents, words of encouragement and push for tenacity ring in my ears

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Abstract:

Review and Update of the Palestinian GNSS Network (1999)

By:

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Ahmed Ghaben **Abed Alaziz Maali**

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Supervisor

Dr. Ghadi Zakarneh

The aim of this project is to conduct a strategic review, update and observe the Palestinian geodetic control framework during 2017. This thesis identifies the issues that should be addressed during that review, and the relative priority of those issues.

Palestinian grid 1923 existing geodetic framework is a reflection of the development needs of the country and the available technology at the time. It is a framework used predominately by surveyors for control of a diversity of projects covering Palestine. These mapping, engineering and infrastructure projects depend on the availability of an accessible, homogeneous statewide geodetic framework with consistent and accurate horizontal and vertical coordinates.

The advent of sub-metre positioning from the GPS satellite constellation has provided real time, active and accurate positioning information to the surveying industry and the wider community. Surveyors still need to relate their GPS position to the existing geodetic framework, while general users calibrate their measurements against the current framework coordinates. Calibration is essential to ensure measurements are on the correct datum and relate to the existing datasets which are based on the Palestinian grid 1923.

GPS technology has advanced to the stage where horizontal and vertical position can be determined to a high accuracy through a network of Continuously Operating Reference Stations (CORS). Future satellite systems to be launched over the next decade offer continuity of services and improved real-time positional information. The USA, Canada, Great Britain, Europe have operational CORS networks, while These countries are maintaining a core framework of ground survey marks from their existing geodetic infrastructure to support survey needs and allow calibration. Uses include asset management, machine control, tectonic plate monitoring, mapping and GIS applications.

Contents

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List of Figures

List of Tables

.

List of Abbreviations

APC	Antenna Phase Center
APREF	Asia-Pacific Reference Frame
ARP	Antenna Reference Point
C/A Code	Coarse Acquisition
CEP	Conventional Ephemeris pole
CIS	Conventional Inertial System
CORS	Continuously Operating Reference Station
CRF	Celestial Reference Frame
CRS	Celestial Reference System
CTP	Conventional Terrestrial Pole
CTS	Coordinated Terrestrial System
DGPS	Differential Global Positioning System
DMA	United States Defense Mapping Agency
DoD	US Department of Defense
ECEF	Earth-Centered, Earth-Fixed
ECI	Earth centered inertial
ETRF	European Terrestrial Reference Frame
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
IAG	International Association of Geodesy
IAU	International Astronomical Union
ICRS	International Celestial Reference System
IERS	International Earth Rotation Service
IGS	International GNSS Service
ITRF	International Terrestrial Reference Frame
ITRS	International Terrestrial Reference System
JPL	NASA's Jet Propulsion Laboratory
LADGPS	Local Area Global Positioning System
NAD83	North American Datum 1983
NGIA	National Geospatial -Intelligence Agency

List of Abbreviations

NNR	no-net-rotation
NRCan	Natural Resources Canada
PMM	Plate Motion Model
PPP	Precise Point Positioning
PPS	Precise Positioning Service
PRN	Pseudo Random Noise
RTK	Real-time, kinematic
SA	Selective Availability
SIRGAS	Sistema de Referencia Geocentrico para las America
SLR	Satellite Laser Ranging
SNR	Signal-to-Noise Ratio
SPS	Standard Positioning Service
TBC	Trimble Business Center
TRS	Terrestrial Reference System
UNB	University of New Brunswick
UTM	Universal Transverse Mercator
VLBI	Very Long Baseline Interferometry
WADGPS	Wide Area Global Positioning System
WGS84	World Geodetic System of 1984

Chapter 1 - Introduction

1.1 Introduction

In space geodetic positioning, where the observation techniques provide absolute positions with respect to a consistent terrestrial reference frame, the corresponding precise definition and realization of terrestrial and inertial reference systems is of fundamental importance. Thanks to significant improvements in receiver technology, to extension and densification of the global tracking network along with more accurate determination of positions and velocities of the tracking stations and to dramatically improved satellite orbits, GPS is today approaching 0.1 ppm precision for longer baselines and it can be considered to be the main global geodetic positioning system providing nearly instantaneous three-dimensional position at the cm accuracy level. One of the fundamental goals of geodesy is to precisely define positions of points on the surface of the Earth, so it is necessary to establish a well-defined geodetic datum for geodetic measurements and positioning computations. Recently, a set of the coordinates established by using GPS and referred to an international terrestrial reference frame could be used as a three-dimensional geocentric reference system for a country.

In the classical sense, a geodetic datum is a reference surface, generally an ellipsoid of revolution of adopted size and shape, with origin, orientation, and scale defined by a geocentric terrestrial frame. Once an ellipsoid is selected, coordinates of a point in space can be given in Cartesian or geodetic (curvilinear) coordinates (geodetic longitude, latitude, and ellipsoid height).

Two types of geodetic datum can be defined namely a static and kinematic geodetic datum. A static datum is thought of as a traditional geodetic datum where all sites are assumed to have coordinates which are fixed or unchanging with time. This is an incorrect assumption since the surface of the earth is constantly changing because of tectonic motion. Static datum does not incorporate the effects of plate tectonics and deformation events. Coordinates of static datum are fixed at a reference epoch and slowly go out of the date, need to change periodically which is disruptive.

Datum's can either become fully kinematic (dynamic), or semi-kinematic. A deformation model can be adopted to enable ITRF positions to be transformed into a static or semi-kinematic system at the moment of position acquisition so that users do not see coordinate changes due to global plate motions. GNSS devices which use ITRF or closely aligned systems position users in agreement with the underlying kinematic frame, however, in practice there are a number of very significant drawbacks to a kinematic datum. Surveys undertaken at different epochs cannot be combined or integrated unless a deformation model is applied rigorously, or is embedded within the data, and the data are correctly time-tagged. On the other hand, Semi–Kinematic datum incorporates a deformation model to manage changes (plate tectonics and deformation events). Coordinates fixed at a reference epoch, so the change to coordinates is minimized. Many countries and regions which straddle major plate boundaries have adopted a semi-kinematic (or semi-dynamic) geodetic datum in order to prevent degradation of the datum as a function of time due to ongoing crustal deformation that is occurring within the country.

High precision GNSS positioning and navigation is very rapidly highlighting the disparity between global kinematic reference frames such as ITRF and WGS84, and traditional static geodetic datum. The disparity is brought about by the increasingly widespread use of PPP and the sensitivity of these techniques to deformation of the Earth due to plate tectonics. In order for precision GNSS techniques to continue to deliver temporally stable coordinates within a localized reference frame.

1.2 Statement of the Problem

Palestine have four map projections and coordinate systems used, these are: Palestine_1923 Grid, Palestine_1923 Belt, Palestine_1923 CS Israel Grid and Israel_TM_Grid, where each coordinate system has its own projection parameters, in this project the Palestine_1923 Grid is the Search topic. these coordinates system were directly used in the classical surveying. But in the modern form of surveying, GNSS became the used method it uses WGS84 as are reference system to integrate GNSS with the classical survey, the coordinates of GNSS to be transform to the local coordinates in Palestine.

More than three different GNSS services currently conflict on using GNSS due the different transformation methods and parameters. The deep-rooted Palestinian national Network need to update the transformation method as well its parameters To get rid of the inconsistencies and unify GNSS provider.

1.3 Objectives of the Thesis

- To study the situation of the Palestine_1923 Grid Network after passing more than 94 years, a check for existing Palestine_1923 Grid.
- To see the quality of the Palestine_1923 Grid networks solution, the selected GPS stations were estimated in its International Terrestrial Reference Frame (ITRF) at the day of the observing campaign and site velocities given by the International Earth Rotation Service (IERS) and then transformed to the original processed ITRF datum, namely ITRF1999.
- To perform the required transformation processing, PPP GPS processing techniques was utilized in the transformation process as well as a three parameters kinematic rigid plate model.
- To push Responsible authorities for maintenance of networks that represent the references (datum)
- To push the inevitability of updating of Palestine_1923 Grid according to the latest frame by taking a modern observation to them and analyze it by PPP

1.4 Literature Review

The transformation of coordinate system had been a case of study from a while, and we are trying to develop this study, a graduate students have researched this before: Somia Zahdeh and Manar Jabari's project made a software that transforms the coordinates from geographic system to geocentric system, and from geocentric to geocentric in different references, and they used 2D-affined, Molodensky, Helmert and others in 2008, Abdallah Radwan (in 2016) developed this software and focused on Molodensky, In 2013 a research has been made by Salem that compared between the Palestinian system and the International Terrestrial Reference Frame (ITRF). Hadi Waddah Dwekat project compare between the geodetic datum transformation method from WGS84 geocentric coordinate as measured using Global navigation sat system to the local coordinates system of Palestine (Palestine_1923).

By applying some of the information of these previous studies, we are going to improve our research to Review, update and observe the Palestinian Geodetic Network of the year 1999 (IRTF99) and execute the necessary calculations In order to be used in the transformation.

1.5 Scope of Thesis

In addition to this chapter, the thesis consists of four chapters as follows:

1.5.1 Chapter Two: Introduction to GPS Measurements. This chapter contains GPS Overview and the principle of operation, segments of the system, error of system, GPS Observation Equations, Relative Positioning Modes, The Precise Point Positioning "PPP", Mathematical Model of Precise Point Positioning, and adjustment of GPS processing.

1.5.2 Chapter Three: Introduction to Geodetic Datums, this chapter discusses the different types of Geodetic datums and their relation to Modern Geodesy and ITRS/ITRF, World Geodetic System (WGS84). It also explains the datum problem, Kinematic Transformation Parameters Using Rigid Plate Rotation Models.

1.5.3 Chapter Four: Introduction to Geodetic Control Networks, this chapter deals with Geodetic Control Networks, Geodetic Horizontal Network Standards, GPS Networks, Control Survey Requirements, Network Design, and Introduction to Network Adjustments.

1.5.4 Chapter Five: This chapter explains how data were collected and analyzed and as well as discussing and evaluating of the obtained results,

1.5.5 Chapter six: Conclusions and Recommendations: This chapter contains the Main conclusions that were derived from the research work and recommendations for the future study.

2.1 GPS Overview

The Global Positioning System (GPS) is an all-weather, space-based navigation system. This system is development by the US Department of Defense (DoD) to satisfy the requirements of the military forces to accurately determine their position, velocity and time in a common reference system, anywhere on or near the earth on a continuous basis.

2.1.1 The principle of operation

The GPS satellites transmit radio signals giving the position of each satellite and the time of transmitting the signal. These signals can be received on the earth with a receiver. The distance between a satellite and the receiver can be computed by multiplying the speed of light with difference between the times that the signal left the satellite and the time that it arrives at the receiver. If the distances to four or more satellites are measured simultaneously, then a three-dimensional position on the earth can be determined. GPS positioning capability is provided at no cost to civilian and commercial users world-wide at an accuracy level of 100 meters. This accuracy level is known as the Standard Positioning Service (SPS). The US military and its allies, and other authorized users, receive a specified accuracy level of 21 meters, known as the Precise Positioning Service (PPS). The full accuracy capability of GPS is denied to users of the SPS through a process known as Selective Availability (SA). This purposeful degradation in GPS accuracy is accomplished by intentionally varying the precise time of the clocks on board the satellites and by providing incorrect orbital positioning data in the GPS navigation message. SA is normally set to a level that will provide 100-meter positioning accuracy to users of the Standard Positioning Service. In practice, several additional sources of error other than SA can affect the accuracy of a GPS-derived position. They include unintentional clock and ephemeris errors, errors due to atmospheric delays, multipath errors, errors due to receiver noise, and errors due to poor satellite geometry [1] .

2.1.2 The segments of the system

The GPS system consists of three segments, the space segment consisting of satellites which broadcast signals, the control segment managing the whole system, and the user segment including the all types of receivers which receive the satellite signals.

Space Segment

The Space segment of the system consists of the GPS satellites. These satellites send radio signals from space. The GPS operational constellation consists of 24 satellites: 21 navigational satellites and 3 active spares orbit the earth in 12 hour orbits as shown in figure.

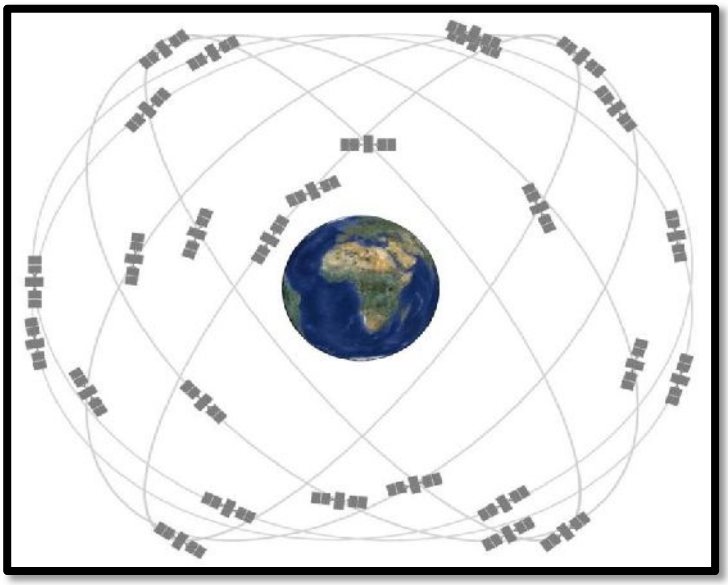

Figure 1 (2.1.): The GPS satellite constellation

These orbits repeat the same ground track (as the earth turns beneath them) once each day. The orbit altitude is such that the satellites repeat the same track and configuration over any point approximately every 24 hours (4 minutes earlier each day). This is accomplished by satellites in nearly circular orbit with an altitude of about 20200 km above the earth. There are six orbital planes (with nominally four satellites in each), equally spaced (60 degrees apart), and inclined at about fifty-five degrees with respect to the equatorial plane. This constellation provides the user with between five and eight satellites visible from any point on the earth. The GPS satellites provide a platform for radio transceivers, atomic clocks, computers and various equipment used for positioning requirements. The equipment of the satellites allows the user to operate a receiver to measure simultaneously distances to more than three satellites. Each satellite broadcasts a message which allows the user to determine the spatial position of the satellite [2].

The satellites in the GPS constellation are arranged into six equally-spaced orbital planes surrounding the Earth. Each plane contains four "slots" occupied by baseline satellites. This 24-slot arrangement ensures users can view at least four satellites from virtually any point on the planet. The Air Force normally flies more than 24 GPS satellites to maintain coverage whenever the baseline satellites are serviced or decommissioned. The extra satellites may increase GPS performance but are not considered part of the core constellation [3].

In June 2011, the Air Force successfully completed a GPS constellation expansion known as the "Expandable 24" configuration. Three of the 24 slots were expanded, and six satellites were repositioned, so that three of the extra satellites became part of the constellation baseline. As a result, GPS now effectively operates as a 27-slot constellation with improved coverage in most parts of the world [2].

Control Segment

The Control Segment consists of a system of tracking stations located around the world. The control segment monitors the functioning of the satellites and uploads orbital, clock correction, and auxiliary data into the satellite memories. This segment consists of two main parts, GPS Master Control, and Monitor Network. The Master Control facility is located at Falcon Air Force Base in Colorado. The monitor stations measure signals from the satellites which are incorporated into orbital models for each satellite. The models compute precise orbital data (ephemeris) and satellite clock corrections for each satellite. The Master Control station uploads ephemeris and clock data to the satellites. The satellites then send subsets of the orbital ephemeris data to GPS receivers over radio signals [3].

User Segment

The GPS User Segment consists of all the GPS receivers and the user community. GPS receivers convert satellite signals into position, velocity, and time estimates. Four satellites are required to compute the four dimensions of position (X, Y, Z) and time. GPS receivers are used for navigation, positioning, time dissemination, and other research. Navigation in three dimensions is the primary function of GPS. Navigation receivers are made for aircraft, ships, ground vehicles, and for hand carrying by individuals. Precise positioning is possible using GPS receivers at reference locations providing corrections and relative positioning data for remote receivers. Surveying, geodetic control, and plate tectonic studies are examples. Time and frequency dissemination, based on the precise clocks on board the satellites and controlled by the monitor stations, are another use for GPS. Astronomical observatories, telecommunication facilities, and laboratory standards can be set to precise time signals or controlled to accurate frequencies by special purpose GPS receivers [3].

2.1.3 The satellite signals

The satellite transmits two microwave carrier signals. These signals are L1 frequency (1575.42 MHz) and L2 frequency (1227.60 MHz). The C/A Code (Coarse Acquisition) modulates the L1 carrier phase. The C/A code is a repeating 1 MHz Pseudo Random Noise (PRN) Code. This noise-like code modulates the L1 carrier signal, "spreading" the spectrum over a 1 MHz bandwidth. The C/A code repeats every 1023 bits (one millisecond). There is a different C/A code PRN for each satellite. GPS satellites are often identified by their PRN number, the unique identifier for each pseudo-random-noise code. The C/A code that modulates the L1 carrier is the basis for the civil SPS. The P-Code (Precise) modulates both the L1 and L2 carrier phases. The P-Code is a very long (seven days) 10 MHz PRN code. In the Anti-Spoofing (AS) mode of operation, the P-Code is encrypted into the Y-Code. The encrypted Y-Code requires a classified AS module for each receiver channel and is for use only by authorized users with cryptographic keys. The P (Y)-Code is the basis for the PPS. The Navigation Message also modulates the L1-C/A code signal. The Navigation Message is a 50 Hz signal consisting of data bits that describe the GPS satellite orbits, clock corrections, and other system parameters. The GPS navigation message consists of time-tagged data bits marking the time of transmission of each sub frame at the time they are transmitted by the satellite. A data bit frame consists of 1500 bits divided into five 300-bit sub frames. A data frame is transmitted every thirty seconds. Three six-second sub frames contain orbital and clock data. Satellite clock corrections are sent in sub frame one and precise satellite orbital data sets (ephemeris data parameters) for the transmitting satellite are sent in sub frames two and three. Sub frames four and five are used to transmit different pages of system data. An entire set of twenty-five frames (125 sub frames) makes up the complete Navigation Message that is sent over a 12.5 minute period. Data frames (1500 bits) are sent every thirty seconds. Each frame consists of five sub frames. Data bit sub frames (300 bits transmitted over six seconds) contain parity bits that allow for data checking and limited error correction. Navigation clock data parameters describe the satellite clock and its relationship to GPS time. Ephemeris data parameters describe satellite orbits for short sections of the satellite orbits. The ephemeris parameters are used with an algorithm that computes the satellite position for any time within the period of the orbit described by the ephemeris parameter set [3].

2.1.4 The Biases

The GPS measurements are affected by both systematic errors and random noise. The systematic errors can be modeled or eliminated by appropriate combinations of the observables as will be explained in section 1.3.6 and 1.4.2. The systematic error sources may be classified into three groups namely satellite related errors, propagation medium related errors, and receiver related errors. The satellite related errors are the clock bias and the orbital errors. The ionospheric and the tropospheric refraction are the propagation medium related error. The antenna phase center variation and the clock bias are considered the receiver related errors. The propagation medium related error specially the ionospheric refraction is considered in this research.

The random noise contains mainly the observation noise and the multipath effects. Multipath is interference between the direct and reflected signals. Multipath is difficult to detect and sometimes hard to avoid. The multipath effect can be considerably reduced by selecting sites protected from reflections and by an appropriate antenna design [3].

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Ionospheric refraction effect

The ionosphere is the part of the earth's atmosphere containing free electrons. This part extends from about 50 to 1000 km above the surface of the earth. The ionosphere is considered as a dispersive medium for the GPS radio signals The vertical ionospheric delay can be written as [4]:

$$\Delta_v ion = \frac{40.3}{f^2} TEC \quad (2.1)$$

Where

$\Delta_v ion$ The vertical ionospheric delay in range units.

f The frequency of the signal.

TEC The total electron content.

The total electron content is a complicated quantity because it depends on the sunspot activities, seasonal and diurnal variations, the line of sight which includes elevation and azimuth of the satellite, and the position of the observation site. The total electron content may be measured, estimated, or eliminated.

The tropospheric effect

Troposphere is the lower part of the earth's atmosphere. It extends from the surface of the earth to about 40 km. The troposphere is nondispersive for frequencies below 30 GHz. Therefore, the propagation of GPS signal in the troposphere is frequency-independent and has the same effect on the phase and the code measurements. The elimination of the tropospheric refraction by dual frequency methods is not possible, so the tropospheric delay should be modeled.

There are various models developed to compute the tropospheric refraction. These models differ primarily with respect to the assumptions made on the vertical refractivity profile and the mapping of the vertical delay with elevation angle. In this research the models of Hopfield, Saastamoninen, and the modified Hopfield models [4] have been used.

2.1.5 GPS Observation Equations

Two different models for the GPS observations can be applied: one model for the code measurements and the other model for phase measurements.

The code observation is the difference between the transmission time of the signal from the satellite and the arrival time of that signal at the receiver multiplied by the speed of light. The time difference is determined by comparing the replicated code with the received one. The time difference is the time shift essential to align these two codes. The code observation represents the geometric distance between the GPS satellite and the receiver plus the bias caused by the satellite and the receiver clock offsets. Moreover, the atmospheric bias and the noise influence the code observations. The basic observation equation related to the code measurement of receiver a to satellite j can be written as:

$$R_a^j(t) = \rho_a^j(t) + C\delta^j(t) - C\delta_a(t) + \Delta_a^j Ion(t) + \Delta_a^j Trop(t) + \zeta \qquad (2.2)$$

Where:

$R_a^j(t)$	The code observation in meter.
$\rho_a^j(t)$	The range between the receiver at station a and satellite j.
C	Speed of light.
$\delta^j(t)$	The bias of the satellite clock.
$\delta_a(t)$	The bias of the receiver clock.
$\Delta^j\ Ion(t)$	The ionospheric delay in meter.
$\Delta^j\ Trop(t)$	The tropospheric delay in meter.
ζ	The noise of the code measurement.

The phase measurement is the difference between the generated carrier phase signal in the receiver and the received signal from the satellite. The phase measurement is in range units when it is multiplied by the signal wave length. It represents the same range and biases as the code observation, and additionally the range related to the unknown integer ambiguities. The observation equation for the phase measurement can be written as the following:

$$\varphi_a^j(t) = \frac{1}{\lambda}\rho_a^j(\text{t}) + N_a^j + \frac{1}{\lambda}C\delta^j(t) - \frac{1}{\lambda}C\delta_a(t) - \frac{1}{\lambda}\Delta_a^j Ion(t) + \frac{1}{\lambda}\Delta_a^j Trop(t) + \varepsilon \qquad (2.3)$$

The above equation can be modified to

$$\varphi_a^j(t) = \frac{1}{\lambda}\rho_a^j(\text{t}) + N_a^j + f\delta^j(t) - f\delta_a(t) - \frac{1}{\lambda}\Delta_a^j Ion(t) + \frac{1}{\lambda}\Delta_a^j Trop(t) + \varepsilon \qquad (2.4)$$

Where:

$\varphi^j(t)$	The phase measurements.
^{j}	The unknown integer ambiguity.
λ	The signal wave length.
f	The signal frequency.
ε	The noise of the phase measurements.

The ionospheric effect has the same absolute value for the code and phase measurements but the signs are opposite. This behavior is due to the different propagation modes for the code and the carrier phase [1] .The code and the phase observation equation are valid for L1 and L2 signals.

2.1.6 Relative Positioning Modes

Relative positioning aims at the determination of the vector between two stations often called a baseline. The coordinates of one of those stations are known with very high accuracy. Relative positioning techniques are always used to eliminate or at least minimize the influence of the involved systematic biases. Relative positioning is an observation technique based on using more than one observing station at the same time rather than relying on the point positioning mode. The errors that influence GPS signals can be greatly reduced or removed using difference modes. Difference modes are much successful for short baselines, as a result from the existing correlation between signals received at several stations simultaneously tracking the same satellites. Difference modes can be performed either on code or carrier phase observations. The relative positioning can be executed between receivers, between satellites, or between epochs, as well as any combination among them leading to single-differences, double-differences, and triple-differences.

Single-difference mode

The single-difference mode is executed between a pair of receivers and one satellite as shown in figure 2.2**.** [5]

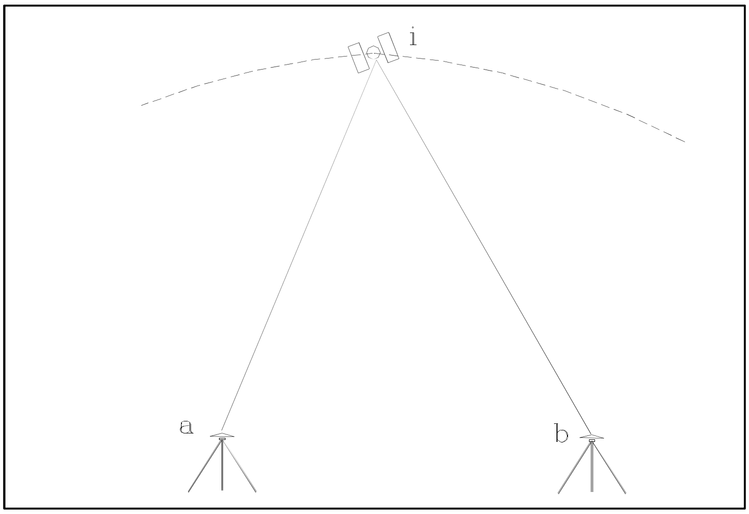

Figure (2.2.): The single-difference technique

Denoting the stations by a and b and the satellite by i. The zero-difference model for phase observations can be written as: equation (2.5) [5]

$$\phi_a^i(t) - f^i\delta^i = \frac{1}{\lambda}\rho_a^i(t) + N_a^i - f^i\delta_a(t) - \frac{1}{\lambda}\Delta_a^i Ion(t) + \frac{1}{\lambda}\Delta_a^i Trop(t)$$

$$\phi_b^i(t) - f^i\delta^i = \frac{1}{\lambda}\rho_b^i(t) + N_b^i - f^i\delta_a(t) - \frac{1}{\lambda}\Delta_b^i Ion(t) + \frac{1}{\lambda}\Delta_b^i Trop(t) \quad (2.5)$$

The difference of the two equations is:

$$\phi_a^i(t) - \phi_b^i(t) = \frac{1}{\lambda}\left[\rho_a^i(t) - \rho_b^i(t)\right] + \left[N_a^i - N_b^i\right] - f^i[\delta_b(t) - \delta_a(t)] - \frac{1}{\lambda}[\Delta_b^i Ion(t) - \Delta_a^i Ion(t)] + \frac{1}{\lambda}[\Delta_a^i Trop(t) - \Delta_b^i Trop(t)] \quad (2.6)$$

By using the shorthand notations:

$$\phi_{a,b}^i(t) = \frac{1}{\lambda}\rho_{a,b}^i(t) + N_{a,b}^i - f^i\delta_{a,b}(t) - \frac{1}{\lambda}\Delta_{a,b}^i Ion(t) + \frac{1}{\lambda}\Delta_{a,b}^i Trop(t) \quad (2.7)$$

The single-difference removes the effect of the satellite clock offset and reduces the effect of the satellite orbital error depending on the distance between the stations. The atmospheric delay is significantly reduced especially with short base lines and can be neglected. In this case the single-difference model for both L1 and L2 frequencies can be written as:

$$\phi_{a,b}^i(t) = \frac{1}{\lambda}\rho_{a,b}^i(t) + N_{a,b}^i - f^i\delta_{a,b}(t) \quad (2.8)$$

Double-difference mode

The double-difference mode is executed between a pair of receivers and pair of satellites as shown in figure (2.3) Denoting the stations by a and b and the satellites to be involved by j, k. Two single-differences according to Equation (2.8) can be applied: [5].

$$\phi^{i}_{a,b}(t) = \frac{1}{\lambda}\rho^{i}_{a,b}(t) + N^{i}_{a,b} - f^{i}\delta_{a,b}(t)$$

$$\phi^{k}_{a,b}(t) = \frac{1}{\lambda}\rho^{k}_{a,b}(t) + N^{k}_{a,b} - f^{k}\delta_{a,b}(t) \quad (2.9)$$

These single-differences are subtracted to get the double-difference model as:

$$\phi^{j}_{a,b}(t) - \phi^{k}_{a,b}(t) = \frac{1}{\lambda}\rho^{j}_{a,b}(t) - \frac{1}{\lambda}\rho^{k}_{a,b}(t) + N^{j}_{a,b} - N^{k}_{a,b} \quad (2.10)$$

By using the shorthand notations:

$$\phi^{j,k}_{a,b}(t) = \frac{1}{\lambda}\rho^{j,k}_{a,b}(t) + N^{j,k}_{a,b} \quad (2.11)$$

The result of this mode is the omission of the receiver clock offsets. The double-difference model for long baselines when there is a significant difference in the atmospheric effect between the two baselines ends can be expressed as:

$$\phi^{i,k}_{a,b}(t) = \frac{1}{\lambda}\rho^{j,k}_{a,b}(t) + N^{j,k}_{a,b} - \frac{1}{\lambda}\Delta^{j,k}_{a,b}Ion(t) + \frac{1}{\lambda}\Delta^{i,k}_{a,b}Trop(t) \quad (2.12)$$

The double-difference model is the applied technique in this research.

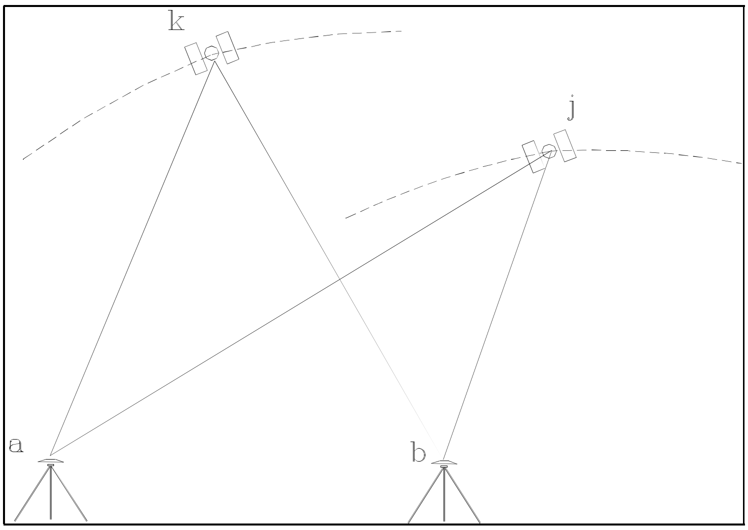

Figure (2.3.): The double-difference technique.

Triple-difference mode

The triple-difference mode is the change in the double-difference observable between two epochs as shown in figure 2.4. Denoting two epochs by t_1,t_2 , the double-difference model at each epoch is : [5].

$$\phi_{a,b}^{j,k}(t_1) = \frac{1}{\lambda}\rho_{a,b}^{j,k}(t_1) + N_{a,b}^{j,k}$$

(2.13)

$$\phi_{a,b}^{j,k}(t_2) = \frac{1}{\lambda}\rho_{a,b}^{j,k}(t_2) + N_{a,b}^{j,k}$$

The corresponding triple-difference equation can be written as

$$\phi_{a,b}^{j,k}(t_1, t_2) = \frac{1}{\lambda}\rho_{a,b}^{j,k}(t_1, t_2)$$

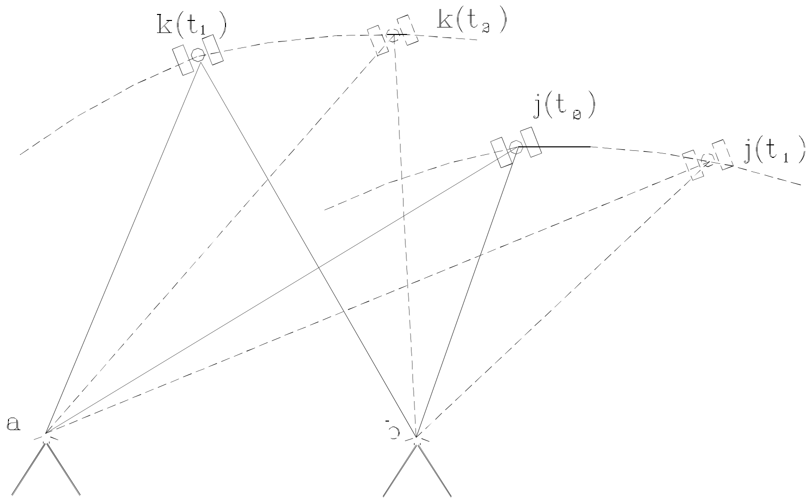

Figure (2.4.): The triple-difference technique

In the above equation, it is assumed that the ambiguity remained unchanged within the time. Therefore, the phase ambiguity bias is canceled. This is true if the receive did not loose lock within the time and no cycle slip occurred.

2.1.7 Linear Combinations

The actual GPS observables are the carrier phases observations ϕ_{L1} , ϕ_{L2} and the code observations R_{L1}, R_{L2}. Some other artificial observations can be created from the actual observations by linearly combining them. The main applied linear combinations in this research will be explained. Any phase combination [5] can be expressed as:

$$\phi_{a,b} = a\phi_1 + b\phi_2 \qquad (2.15)$$

The corresponding frequency will be

$$f_{a,b} = a f_1 + b f_2 \qquad (2.16)$$

The noise level is also affected by the linear combination. In this case the standard deviation can be written in range units as:

$$\sigma_{a,b} = \lambda_{a,b}\sqrt{a^2\sigma_1^2 + b^2\sigma_2^2} \qquad (2.21)$$

Where:

σ_1, σ_2

The standard deviation in L1 and L2 observations in cycles respectively.

If the noise in L1 and L2 observations have the same standard deviation in cycles, the ratio between the noise level in the linear combinations and L1 observation can be written as:

$$\chi_{noise} = \frac{\lambda_2\sqrt{a^2 + b^2}}{a\,\lambda_2 + b\lambda_1} \qquad (2.22)$$

Where:

χ_{noise} The noise ratio.

Table (2.1.): The characteristics of the used linear combinations.

Signal	a	B	$\lambda_{a,b}$ (m)	ψ_{ion}	χ_{noise}
L1	1	0	0.190	1.00	1
L2	0	1	0.244	1.65	1.3
Wide-lane	1	-1	0.862	-1.28	6.4
Narrow-lane	1	1	0.107	1.28	0.8
Ionosphere-free	77	-60	0.006	0.00	3.2
low ionospheric effect	5	-4	0.101	-0.71	3.4
Very long wave length	-7	9	14.65	350.35	877.9

Some different linear combinations are applied in this research. Table 2.1 shows the characteristics of these linear combinations [5].

2.1.8 Wide-lane and narrow-lane linear combinations

There are two linear combinations which play an important role in the fixation of the unknown ambiguities, namely the wide-lane and the narrow-lane linear combination. The wide-lane linear combination can be expressed of L1 and L2 phase observations as:

$$\phi_w = \phi_1 - \phi_2 \qquad (2.23)$$

and the narrow-lane linear combination as:

$$\phi_n = \phi_1 + \phi_2 \qquad (2.24)$$

The wave length of the wide-lane combination is about 86 cm which is approximately 4

times the wave length of L1 or L2 observations as shown in table 1.1. This means that the ambiguity resolution process is generally much simpler when using such a combination than using L1 or L2 observations.

There is some advantage in using the wide-lane and the narrow-lane linear combination in the ambiguity resolution process. There is an even odd relation between the wide-lane and the narrow-lane ambiguities. When the wide-lane ambiguity is odd the corresponding narrow-lane ambiguity has also to be odd, similarly when the wide-lane ambiguity is even the corresponding narrow-lane ambiguity has to be even. The even odd relation implies that when one of the ambiguities of these combinations is firstly resolved the effective wave length of the other combination will be increased by a ratio of 2; consequently, it can be resolved more easily.

2.1.9 Ionosphere-free linear combinations

The ionosphere-free is another important linear combination that used in this investigation. To eliminate the effect of the ionospheric refraction a linear combination between two signals with different frequencies is used. The ionosphere-free linear combination of L1 and L2 phase observations can be expressed as:

$$\phi_{ion} = \phi_1 - \frac{f}{1}\phi_2 \qquad (2.25)$$

The ionosphere-free linear combination has a significant disadvantage because the resulting ambiguity is no longer an integer. This combination can be written in other form as:

$$\phi_{ion} = 77\phi_1 - 60\phi_2 \qquad (2.26)$$

The left- h a n d side of Equation (2.25) and Equation (2.26) are different. The above relation cannot be applied for ambiguity resolution because the wave length is very short, about 0.63 cm. Such short wave length makes the ambiguity resolution practically impossible. The estimated position using the ionosphere-free combination after resolving the ambiguities is not influenced by the ionospheric effect. The elimination of the ionospheric refraction is the huge advantage of this combination. This method is the main reason the GPS signal has two carrier waves L1 and L2 [1].

2.1.10 The Mathematical Model for Relative Positioning

The double -difference is selected for treatment in detail, equation (2-44). The canceling effect of the receiver clock biases is the reason double differences are preferably used. This cancellation resulted from the assumptions of simultaneous observations and equal frequencies of the satellite signals. The final form of the double difference equation is:

$$\varphi_{ab}^{jk}(t) = \frac{1}{\lambda} R_{AB}^{jk}(t) + N_{AB}^{jk} \qquad (2\text{-}49)$$

The model for the double –difference of equation (2-48), multiplied by , is

$$\lambda\ \varphi_{ab}^{jk}(t) = R_{AB}^{jk}(t) + \lambda\ N_{AB}^{jk} \qquad (2-50)$$

Where the term: $R^{jk}(t)$, containing the geometry , is composed as

$$R_{AB}^{jk}(t) = R_{B}^{k}(t) - R_{B}^{j}(t) - R_{A}^{k}(t) + R_{A}^{j}(t) \qquad (2-51)$$

which reflects the fact of four measurement quantities for a double –difference.

Each of four terms must be linearized according to [1] yielding

$$R_{AB}^{jk}(t) = R_{B0}^{k}(t) - \frac{x^{k}(t) - x_{B0}}{R_{B0}^{k}(t)}\Delta X_B - \frac{Y^{k}(t) - Y_{B0}}{R_{B0}^{k}(t)}\Delta Y_B - \frac{Z^{k}(t) - Z_{B0}}{R_{B0}^{k}(t)}\Delta Z_B - R_{B0}^{J}(t)$$

$$+ \frac{x^{j}(t) - x_{B0}}{R_{B0}^{j}(t)}\Delta X_B - \frac{Y^{j}(t) - Y_{B0}}{R_{B0}^{j}(t)}\Delta Y_B - \frac{Z^{j}(t) - Z_{B0}}{R_{B0}^{j}(t)}\Delta Z_B - R_{A0}^{k}(t) + \frac{X^{k}(t) - Z_{A0}}{R_{A0}^{k}(t)}\Delta X_A$$

$$+ \frac{Y^{k}(t) - Y_{A0}}{R_{A0}^{k}(t)}\Delta Y_A - \frac{Z^{k}(t) - Z_{A0}}{R_{A0}^{k}(t)}\Delta Z_A - R_{A0}^{J}(t) - \frac{X^{j}(t) - X_{A0}}{R_{A0}^{j}(t)}\Delta X_A - \frac{Y^{j}(t) - Y_{A0}}{R_{A0}^{j}(t)}\Delta Y_A$$

$$- \frac{Z^{j}(t) - Z_{A0}}{R_{A0}^{j}(t)}\Delta Z_A \qquad (2\text{-}52)$$

Substituting (2-51) into (2-50) and rearranging leads to the linear observation equation

$$R_{AB}^{jk}(t) = a_{XA}^{jk}(t)\Delta X_A + a_{YA}^{jk}\Delta Y_A + a_{ZA}^{jk}\Delta Z_A + a_{XB}^{jk}(t)\Delta X_B + a_{YB}^{jk}\Delta Y_B + a_{ZB}^{jk}\Delta Z_B + \lambda N_{AB}^{jk} \qquad (2\text{-}52)$$

Where the left side

$$^{jk}(t) = \lambda\ \varphi^{jk}(t) - R^{k}(t) + R^{j}(t) + R^{k}(t) - R^{j}(t) \qquad (2\text{-}54)$$

Comprising both the measurement quantities and all terms computed from the approximate values. On the right side of (2-53), the abbreviations have been used

$$a_{XA}^{jk}(t) = +\frac{X^k(t) - X_{A0}}{R_{A0}^k(t)} - \frac{X^j(t) - X_{A0}}{R_{A0}^j(t)}$$

$$a_{YA}^{jk}(t) = +\frac{Y^k(t) - Y_{A0}}{R_{A0}^k(t)} - \frac{Y^j(t) - Y_{A0}}{R_{A0}^j(t)}$$
$$a_{ZA}^{jk}(t) = +\frac{Z^k(t) - Z_{A0}}{R_{A0}^k(t)} - \frac{Z^j(t) - Z_{A0}}{R_{A0}^j(t)} \quad (2\text{-}55)$$
$$a_{XB}^{jk}(t) = -\frac{X^k(t) - X_{B0}}{R_{B0}^k(t)} - \frac{X^j(t) - X_{B0}}{R_{B0}^j(t)}$$

$$a_{YB}^{jk}(t) = -\frac{Y^k(t) - Y_{B0}}{R_{B0}^k(t)} - \frac{Y^j(t) - Y_{B0}}{R_{B0}^j(t)}$$
$$a_{ZB}^{jk}(t) = -\frac{Z^k(t) - Z_{B0}}{R_{B0}^k(t)} - \frac{Z^j(t) - Y_{B0}}{R_{B0}^j(t)}$$

Note that the coordinates of one point (e.g. A) must be known for relative positioning. More specifically, the known point A reduces the number of unknowns by three because of :

$$\Delta X_A = \Delta Y_A = \Delta Z_A = 0 \quad (2\text{-}56)$$

And leads to slight change in the left side term

$$L_{AB}^{jk}(t) = \lambda\varphi_{AB}^{jk}(t) - R_{B0}^k(t) + R_{B0}^j(t) + R_A^k - R_A^j(t) \quad (2\text{-}57)$$

Assuming now four satellites j ,k ,l ,m and two epochs , the matrix- vector system

$$L = \begin{bmatrix} l_{AB}^{jk}(t_1) \\ l_{AB}^{jl}(t_1) \\ l_{AB}^{jm}(t_1) \\ l_{AB}^{jk}(t_2) \\ l_{AB}^{jl}(t_2) \\ l_{AB}^{jm}(t_2) \end{bmatrix} \qquad x = \begin{bmatrix} \Delta X_B \\ \Delta Y_B \\ \Delta Z_B \\ N_{AB}^{jk} \\ l_{AB}^{jl} \\ l_{AB}^{jm} \end{bmatrix} \quad (2\text{-}58)$$

$$A = \begin{bmatrix} a_{XB}^{jk}(t_1) & a_{YB}^{jk}(t_1) & a_{ZB}^{jk}(t_1) & \lambda & 0 & 0 \\ a_{xB}^{jl}(t_1) & a_{YB}^{jl}(t_1) & a_{ZB}^{jl}(t_1) & 0 & \lambda & 0 \\ a_{XB}^{jm}(t_1) & a_{YB}^{jm}(t_1) & a_{ZB}^{jm}(t_1) & 0 & 0 & \lambda \\ a_{XB}^{jk}(t_2) & a_{YB}^{jk}(t_2) & a_{ZB}^{jk}(t_2) & \lambda & 0 & 0 \\ a_{XB}^{jl}(t_2) & a_{YB}^{jl}(t_2) & a_{ZB}^{jl}(t_2) & 0 & \lambda & 0 \\ a_{XB}^{jm}(t_2) & a_{YB}^{jm}(t_2) & a_{ZB}^{jm}(t_2) & 0 & 0 & \lambda \end{bmatrix}$$

is obtained which represents a determined and, thus, solvable system. Note that for one epoch, the system has more unknowns than observation equations.

2.2 The Precise Point Positioning "PPP"

Precise Point Positioning (PPP) is a satellite based positioning technique aiming at highest accuracy in close to real-time. First investigations using dual frequency data

from a single GPS receiver data for a few cm-positioning in post- processing mode have been published in 1997 by JPL. Utilizing the ionosphere free linear combination, the remaining required model information like precise orbits and clocks issued by the IGS has been used. Within the last decade many approaches have been carried out to serve applications in close to real- time by this technique.

Although traditionally a double-differencing processing tool, the Bernese software is also capable of analyzing undifferenced GPS measurements in post processing mode. BSW PPP is very fast and efficient in generating cm-level accuracy station coordinates. Nevertheless, it is not possible to reach a coordinate quality as obtained from a network analysis.

Since PPP is a technique with only one GPS receiver, no differences between two receivers can be built to eliminate satellite specific errors such as clock and orbital errors. Therefore, it is necessary to use the most precise satellite clock corrections and satellite orbits. Relevant products, available even in real-time, are for example IGS ultra rapid precise ephemerides ensuring an orbital representation of 10-15 cm and better than 1.5 ns clock accuracy over a prediction period of 2 hours and more. Beyond that the use of the non-integer ionosphere free linear combinations leads to further effects. The combined code and phase noise is amplified compared to the noise of isolated signals. Furthermore, the integer characteristics of the phase ambiguities get lost and ambiguity fixing is prevented, which leads to even longer convergence times.

Convergence times are the time spans from start to a stably accurate solution. The convergence time to reach decimeter accuracy i s typically about 30 minutes under normal conditions. To reach centimeter accuracies the PPP processor needs significantly longer [6].

In comparison with common techniques like DGPS or RTK, the costs are reduced, because no base stations and no simultaneous observations are necessary. On the other hand the necessary models have to be fetched either from globally acting services like IGS (orbits, satellite clocks) or from regional GNSS service providers (atmospheric delays) and standard interfaces (e.g. RTCM) have to be developed to forward this information to the rover. Further problems still to be solved are coordinate convergence periods of up to 2 hours as well as ambiguity resolution, which are harmed by non-integer calibration phase biases. These biases vanish only in difference mode and must be determined a priori.

PPP also provides a positioning solution in a dynamic, global reference frame such as the International Terrestrial Reference Frame (ITRF) negating any local distortions associated with differential positioning techniques when local coordinates are used at the Continuously Operating Reference Station (CORS). However, it is important to fully understand the implications of transforming between a global and a national or local datum for example, [7].

At present, post-processed PPP offers the most comparable accuracies to Differential GPS (DGNSS) positioning techniques. Free PPP post-processing services such as Auto-GIPSY(http://apps.gdgps.net/) and CSRSPPP (http://www.geod.nrcan .gc.ca/productsproduits/ ppp_e.php) provide converged float solutions at the centimeter-level, thereby allowing PPP to offer a viable alternative to post-processed DGNSS solutions. Users upload their observed RINEX data files to such online services, and the coordinate solution for the (static or kinematic) GNSS receiver's position is computed automatically. Note, however, that long observation session times (several hours) are required to obtain “comparable accuracies”, and therefore the applications are typically restricted to the establishment of geodetic control using GNSS technology [8].

2.2.1 Mathematical Model of Precise Point Positioning

Recall that mainly, there are two types of GPS observables, namely the code pseudo ranges and carrier phase observables. In general, the pseudo range observations are used for coarse navigation, whereas the carrier phase observations are used in high-precision surveying applications. That is due to the fact that the accuracy of the carrier phase observations is much higher than the accuracy of code observations, [9]. The pseudo range observation equations denoted in chapter two can be written again in case of L1, and L2 as [10]:

$$P_{L1} = \rho + c(dt - dT) + d_{ion} + d_{trop} + d_{orb} + \varepsilon_p \qquad (2\text{-}27)$$

$$P_{L2} = \rho + c(dt - dT) + d_{ion} + d_{trop} + d_{orb} + \varepsilon_p \qquad (2\text{-}28)$$

Where:

P_{L1}, P_{L2}	are the observed pseudo range on L1, L2 respectively.
ρ	is the unknown geometric satellite to receiver range.
C	is speed of light
dt, dT	are satellite and receiver clock errors respectively.
d_{ion}, d_{ion}	are the ionosphere error on L1 and L2 respectively.
d_{trop}	is the troposphere error.
d_{orb}	is the orbital error
ε_{PL1}, ε_{PL2}	are the code measurement noise on L1, L2 respectively.

Also, the phase observation equations are:

$$\Phi_{L1} = \rho + c(dt - dT) + \lambda_{L1}.N_{L1} - d_{ion_{L1}} + d_{trop} + d_{orb} + \varepsilon_{\phi_{L1}} \qquad (2\text{-}29)$$

$$\Phi_{L2} = \rho + c(dt - dT) + \lambda_{L2}.N_{L2} - d_{ion_{L2}} + d_{trop} + d_{orb} + \varepsilon_{\phi_{L2}} \qquad (2\text{-}30)$$

Where:

Φ_{L1}, Φ_{L2}	are the observed phase on L1, L2 respectively.
λ_{L1}, λ_{L2}	are the L1, L2 carrier wavelength.
N_{L1}, N_{L2}	are the ambiguities on L1, L2.
$\varepsilon_{\varphi L1}$, $\varepsilon_{\varphi L1}$	are the phase measurement noise on L1, L2 resp ectively.

The GPS single point positioning model GPS-SPP is depending on eliminating the ionospheric error from three linear combinations as follows [10]:

1. The ionosphere-free phase combination consists of multiplying equation (2-29) by $\frac{f_1^2}{f_1^2-f_2^2}$ and equation (2-30) by $\frac{-f_2^2}{f_1^2-f_2^2}$ and sum the two new equations. This gives:

$$\phi_{IF} = \frac{f_1^2}{f_1^2-f_2^2} \cdot \phi_{L1} - \frac{f_2^2}{f_1^2-f_2^2} \cdot \phi_{L2} \qquad (2\text{-}31)$$

Where:

ϕ_{IF}	is the ionosphere-free phase combination.
f_1, f_2	are the L1, L2 carrier frequencies .

2. Summation of equations (2-27), equation(2-28) and equation(2-29) , equation(2-30) and multiplying the sum by 0.5 yield to ionosphere-free code-phase on both L1 and L2, as follows:

$$P_{L1 \; L1} = 0.5(P_{L1} + \Phi_{L1}) \qquad (2\text{-}32)$$

$$P_{P \; +\Phi} = 0.5(P_{L2} + \Phi_{L2}) \qquad (2\text{-}33)$$

At this point the GPS-SPP model consists of three observations for each satellite. Equations (2-31), (2-32)and (2-33) can be rewritten as:

$$\Phi_{IF} = \rho + c(dt - dT) + \frac{f_1^2}{f_1^2-f_2^2}\lambda_1.N_1 - \frac{f_2^2}{f_1^2-f_2^2}\lambda_2.N_2 + d_{trop} + d_{orb} + \varepsilon_{\phi IF}$$

(2-34)

$$P_{P\ L1+\Phi_{L1}} = \rho + c(dt - dT) + d_{trop} + d_{orb} + 0.5\lambda_1.N_1 + 0.5(\varepsilon_{PL1} + \varepsilon_{\phi L1}) \quad (2\text{-}35)$$

$$P_{P\ L2+\Phi_{L2}} = \rho + c(dt - dT) + d_{trop} + d_{orb} + 0.5\lambda_2.N_2 + 0.5(\varepsilon_{PL2} + \varepsilon_{\phi L2}) \quad (2\text{-}36)$$

Applying the IGS products on the above three equations, and use Saastamoinen tropospheric model indicated, lead to removal of satellite clock error, orbital error, and troposphere error. Thus, the equations can be rewritten again as [10]:

$$\Phi_{IF} = \rho + c.dt + \frac{f_1^2}{f_1^2 - f_2^2}\lambda_1.N_1 - \frac{f_2^2}{f_1^2 - f_2^2}\lambda_2.N_2 + \varepsilon_{\phi IF} \quad (2\text{-}37)$$

$$P_{P\ L1+\Phi_{L1}} = \rho + c.dt + 0.5\lambda_1.N_1 + 0.5(\varepsilon_{PL1} + \varepsilon_{\phi L1}) \quad (2\text{-}38)$$

$$P_{P\ L2+\Phi_{L2}} = \rho + c.dt + 0.5\lambda_2.N_2 + 0.5(\varepsilon_{PL2} + \varepsilon_{\phi L2}) \quad (2\text{-}39)$$

Assume we have k satellites, then the observations equations (n) will be 3k, the unknowns (u) will be the 3-D coordinates of the receiver point, the receiver clock error, N1, N2 ambiguities for each satellite. To solve this system of equations $3k \geq 3 + 1 + 2k$ or simply$k \geq 4$. This means that, to solve an epoch by epoch SPP model, at least four satellites must be tracked [10].

2.2.2 Variance Estimation

Typically, GPS observables are pseudo ranges derived from code or phase measurements. The accuracy of code ranges is at the sub-meter level, whereas the accuracy of the carrier phase is in the millimeter range [11].With high- end GPS receivers, the code and phase noise levels are approximately 10cm and 0.3cm respectively. Hence, one can assume the noise level of the pseudo range and phase for both carrier signals as:

$$\sigma_{P_{L1}} = \sigma_{P_{L2}} = \sigma_{P_L} = 10cm \ and \ \sigma_{\Phi_{L1}} = \sigma_{\Phi_{L2}} = \sigma_{\Phi_L} = 0.3cm \quad (2\text{-}40)$$

Applying the variance propagation law, to determine the variance of the ionosphere-free phase combination, one can get:

$$\sigma_{\Phi_{IF}}^2 = \left[\frac{f_1^2}{f_1^2 - f_2^2}\right].\sigma_\phi^2 + \left[\frac{f_2^2}{f_1^2 - f_2^2}\right].\sigma_\phi^2$$

$$\sigma_{\Phi_{IF}}^2 = 6.48.\sigma_\phi^2 + 2.39.\sigma_\phi^2 = 8.87\sigma_\phi^2 \quad (2\text{-}41)$$

Analogously, the variance of the ionosphere-free from code and phase combination after neglecting the phase noise can be deduced from:

$$\sigma^2_{P\ PL1+\Phi_{L1}} = \sigma^2_{P\ L2+\Phi_{L2}} = 0.5^2.\sigma^2_P + 0.5^2.\sigma^2_\Phi = \frac{1}{4}\sigma^2_P \qquad (2\text{-}42)$$

2.2.3 Ambiguity Initialization

As stated before, the unknowns in the GPS-SPP model are the 3-D receiver coordinates, and the receiver clock offset, and the double ambiguities for each satellite. To solve the system of equations and applying the least squares principle, one must have initial values for the unknowns. The approximate values for the 3-D receiver coordinates along with the receiver clock offset are given in the navigation message. On the other hand, the approximate values for the ambiguities unknown are deduced from the following procedure [5]:

1. get approximate values for the ionosphere error on L1, L2 by subtracting equation (2-28) from equation (2-27), this yields to:

$$P_{L1} - P_{L2} = d_{ion_{L1}} - d_{ion_{L2}} \qquad (2\text{-}43)$$

The ionosphere error is inversely proportional to the squaring frequency of the

carrier signal. Hence,

$$d_{ion_{L2}} = \frac{f_1^2}{f_2^2} d_{ion_{L1}} \qquad (2\text{-}44)$$

Substituting equation (2-43) in equation (2-42), we can get an approximate solution for the ionosphere error.

$$d_{ion_{L1}} = \frac{P_{L1} - P_{L2}}{1 - \frac{f_1^2}{f_2^2}} \qquad (2\text{-}45)$$

2. get the approximate values of the ambiguities from subtracting equation (2-27) from equation

(2-29), leads to [5]:

$$\Phi_{L1} - P_{L1} = \lambda_{L1}.N_{L1} - 2d_{ion_{L1}} \qquad (2\text{-}46)$$

Substitute the approximate value of the ionosphere-free from equation (2-43) into equation (2-45), one can get the approximate values of the ambiguities on the L1 carrier from [5]:

$$\lambda_{L1}.N_{L1} = \Phi_{L1} - P_{L1} + \frac{2(P_{L1}-P_{L2})}{\left[1-\frac{f_1^2}{f_2^2}\right]} \qquad (2\text{-}47)$$

The same analysis can be done, to obtain the approximate values on the L2 carrier, and one can get the approximate values of the ambiguities on L2 carrier from [5]:

$$\lambda_{L2}.N_{L2} = \Phi_{L2} - P_{L2} + \frac{2(P_{L2}-P_{L1})}{\left[1-\frac{f_2^2}{f_1^2}\right]} \qquad (2\text{-}48)$$

2.2.4 Least Squares Adjustment

Least squares adjustment is normally used at two different stages in the processing of GPS carrier-phase measurements. First, it applied in the adjustment that yields baseline components between stations from the redundant carrier-phase observations. Recall that in this procedure, differencing techniques employed to compensate for errors in the system and to resolve the cycle ambiguities. In the solution, observation equations contain the differences in coordinates between stations as parameters. The reference coordinate system for this adjustment is the X_e ,Y_e,Z_e geocentric system. A highly redundant system of equations obtained because a minimum of four (and often more) satellites are tracked simultaneously using at least two (and often more) receivers. Furthermore, many repeat observations taken. This system of equations solved by least squares to obtain the most probable ΔX, ΔY, and ΔZ components of the baseline vectors. Software furnished by manufacturers of GPS receivers will process observed phase changes to form the differencing observation equations, perform the least squares adjustment, and output the adjusted baseline vector components. The software will also output the covariance matrix, which expresses the correlation between the ΔX, ΔY, and ΔZ components of each baseline. The second stage where least square employed in processing GPS observations is in adjusting baseline vector components in networks. This adjustment made after the least squares adjustment of the carrier-phase observations is completed [12].

2.2.5 Standard Adjustment

There are numerous adjustment techniques that can be used, but least squares adjustment with parameters is the only one discussed here. It based on equations where the observations expressed as a function of unknown parameters. A Taylor series expansion usually performed in the case of nonlinear functions. This requires approximate values for the parameters. The Taylor series expansion must be truncated after the second term to obtain a linear function with respect to the unknowns. The resulting linear observation model can represent in a matrix-vector notation as [1]:

$$L = Ax \tag{2-59}$$

Where

L is the vector of observations

A is the design matrix

x is the vector of unknowns. By introducing in addition, the definitions

$\sum$ is the covariance matrix,

The cofactor matrix of observations is

$$Q_1 = \frac{1}{\sigma_0^2}\Sigma \quad (2\text{-}60)$$

and

$$P = Q_1^{-1} \quad (2\text{-}61)$$

is the weight matrix. Assuming n observations and u unknown parameters leads to a design matrix A comprising n rows and u columns. For n > u the system Equation(2-58)is over determined and, in general, non-consistent because of observational errors or noise. To assure consistency, the noise vector n is added to the vector of observations and Equation (2-58) thus converts to [1]

$$L + n = Ax \quad (2\text{-}62)$$

The solution of this system becomes unique by the least squares principle (n^T P n)= minimum. The application of this minimum principle on the observation equations (4-35) leads to the normal equations

$$A^T P Ax = A^T P L \quad (2\text{-}63)$$

With the solution

$$x = (A^T P A)^{-1} A^T P L \quad (2\text{-}64)$$

Which can be simplified to

$$x = G^{-1} g \quad (2\text{-}65)$$

where $G = A^T P A$ and $g = A^T P L$

the cofactor matrix Q_x follows from $x = G^{-1} A^T P L$ by the covariance propagation law as

$$Q_x = (G^{-1} A^T P)\, Q_1\, (G^{-1} A^T P)^T \quad (2\text{-}66)$$

and reduces to

$$Q_x = G^{-1} = (A^T P A)^{-1} (2\text{-}67)$$

by substituting $Q_x = P^{-1}$

\

2.2.6 Sequential Adjustment

Assume a partitioning of the observation model Equation (2-61) into two subsets [1]

$$L = \begin{bmatrix} L1 \\ L2 \end{bmatrix} \qquad n = \begin{bmatrix} n1 \\ n2 \end{bmatrix} \qquad A = \begin{bmatrix} A1 \\ A2 \end{bmatrix} \tag{2-68}$$

Using the first set only, a preliminary solution x_o can be calculated according to Equation (2-83) and Equation (2-67)by

$$\begin{pmatrix} X_0=(A_1^T P_1 A_1)^{-1} A_1^T A P_1 L_1 = G_1^{-1} g_1 \\ Q_{X_O}=(A_1^T P_1 A_1)^{-1} = G_1^{-1} \end{pmatrix} \tag{2-69}$$

Provided that there is no correlation between the two subsets of observations, the weight matrix

$$P = \begin{bmatrix} P_1 & \cdots & 0 \\ & & \\ 0 & \cdots & P_1 \end{bmatrix} \tag{2-70}$$

Is a block-diagonal matrix. The matrix G and the vector g for the adjustment of the full set of observations result from adding the corresponding matrices and vectors for the two subsets:

$$\left.\begin{aligned} G = A^T P A = (A_1^T P_1 A_1 + A_2^T P_2 A_2) = G_1 G_2 \\ g = A^T P L = (A_1^T P_1 L_1 + A_2^T P_2 L_2) = g_1 g_2 \end{aligned}\right\} \tag{2-71}$$

If the change of the preliminary solution x_o due to the additional observation set L_2 is denoted as Δx, then

$$(G_1 + G_2)(X_O + \Delta X) = G_1 + G_2 \tag{2-72}$$

Is the appropriate formulation of the adjustment. This equation can slightly rearranged to

$$(G_1 + G_2)\Delta X = g_1 + g_2 - (G_1 + G_2)X_o \tag{2-73}$$

where the right-hand side, cf. Equation (2-72), can be simplified because of the relation $g_l - G_l x_o = 0$ so that

$$(G_1 + G_2)\Delta X = g_2 - G_2 X_o \tag{2-74}$$

Results. Resubstituting from Equation (2-74) $g_2 = A_2^t P_2 B_2$ and G_2 yields

$$(G_1 + G_2)\Delta X = A_2^T P_2 L_2 - A_2^T P_2 A_2 X_o \tag{2-75}$$

or

$$(G_1 + G_2)\Delta X = A_2^T P_2 (L_2 - A_2 X_O) \quad (2\text{-}76)$$

And

$$\Delta X = (G_1 + G_2) A_2^T P_2 (L_2 - A_2 X_O) \quad (2\text{-}77)$$

or finally

$$\Delta X = K(L_2 - A_2 X_O) \quad (2\text{-}78)$$

Where:

$$K = (G_1 + G_2)^{-1} A_2 P_2 \quad (2\text{-}79)$$

Note that formally in Equation(2-78)the term A_2 x_o can be considered as prediction for the observations L_2

The goal of the next step is the computation of the change ΔQ with respect to the preliminary cofactor matrix Q x_o. Starting point is the relation

$$GQx_o = (G_1 + G_2)(Qx_o + \Delta Q) = I \quad (2\text{-}80)$$

where I denote the unit matrix. This equation reformulated as [1]

$$(G_1+G_2)\ \Delta Q = I- (G_1+G_2)\ Q\ x_o \quad (2\text{-}81)$$

and, since G_1 Q x_o = I, this reduces to

$$(G_1+G_2)\ \Delta Q = -\ G_2\ Q\ x_o \quad (2\text{-}82)$$

or

$$\Delta Q = -\ (G_1+G_2)^{-1}\ G_2\ Q\ x_o \quad (2\text{-}83)$$

and, by using $G_2 = A_2^T P_2 A_2$, the relation

$$\Delta Q = -(G_1 + G_2)^{-1} A_2^T P_2 A_2 Q x_O \quad (2\text{-}83)$$

follows, comparing this equation with Equation(2-83)K may be substituted and

$$\Delta Q = -K\ A_2\ Q\ x_o \quad (2\text{-}85)$$

results. Matrix K which is denoted as gain matrix, satisfies the very remarkable relation

$$K = (G_1 + G_2)^{-1} A_2^T P_2 = G_1^{-1} A_2^T (P_2^{-1} + A_2 G_1^{-1} A_2^T)^{-1} \quad (2\text{-}86)$$

It is essential to learn from Equation (2-86) that the first form for K implies the inversion of a u x u matrix if u is the number of unknown parameters; whereas, for the second form an inversion of an n_2 x n_2 matrix is necessary when n_2 denotes the number of observations for the second subset. Therefore, the second form is advantageous as long as $n_2 < u$.

A final remark should conclude the section on the sequential adjustment. In the equation of Δx, cf. Equation(2-82), Equation(2-83), and in the equation for ΔQ, cf. Equation(2-81), Equation (2-83), neither the design matrix A_1 nor the vector L_1 for the first set of observations appears explicitly. Therefore, formally, the substitution e.g. $A_1 = I$ and $L_1 = x_o$ may be performed and the model for the sequential adjustment is then formulated as

$$x_o + n_1 = x$$
$$L_2 + n_2 = A_2\, x. \tag{2-87}$$

Model Equation(2-87) reflects that the preliminary estimates x_o for the unknown parameters are introduced into the sequential adjustment as observations [1].

2.2.7 Epoch by Epoch Least Square Adjustment

The GPS-SPP model is depending on solving an epoch by epoch adjustment solution. As stated before, for k satellites, we have 3k observations equations (n), and 4+2k unknowns (u). In case of 4 satellites, the observations will be 3*4=12, and the number of known will be 3+1+2*4=12. For any number of satellites more than 4, the number of observations will be greater than the number of unknowns, hence the system of equations is an over-determined equation system, which must be solved using the least-squares principles, to get the most reliable values for all the involved unknowns. In this equation system, the observation vector L and the Parameter X can be given as:

$$L = \begin{bmatrix} P_{(P_{L1}+\Phi_{L1})\ sat\#1} \\ P_{(P_{L2}+\Phi_{L2})\ sat\#1} \\ \Phi_{IF_{sat\#1}} \\ P_{(P_{L1}+\Phi_{L1})\ sat\#2} \\ P_{(P_{L2}+\Phi_{L2})\ sat\#2} \\ \Phi_{IF_{sat\#2}} \\ ------- \\ ------- \\ ------- \\ P_{(P_{L1}+\Phi_{L1})\ sat\#n} \\ P_{(P_{L2}+\Phi_{L2})\ sat\#n} \\ \Phi_{IF_{sat\#n}} \end{bmatrix} \quad (2.88)$$	$$X_{u*1} = \begin{bmatrix} X \\ Y \\ Z \\ t \\ N1_{sat\#1} \\ N2_{sat\#1} \\ N1_{sat\#2} \\ N2_{sat\#2} \\ ------ \\ ------ \\ N1_{sat\#n} \\ N2_{sat\#n} \end{bmatrix} \quad (2.89)$$

The parametric least squares adjustment solution becomes in matrix form:

$$A.\delta + W - V = 0 \qquad (2\text{-}90)$$

Where: A is the design matrix, **δ** is the vector of corrections to the unknown parameters X, $W = f(X^0) - L = L^0 - L$ is the misclosure vector, X^0 is the vector of approximate values for unknowns, and V is the vector of residuals. L is the vector of observations.

The coefficient matrix A, which is defined as the first derivative of the involved observation equations with respect to the unknowns, can be formed by a direct differentiation

of the observation equations (2.88) with respect to the involved unknowns, given by equation (2- 89). The number of the rows in matrix A will be the same as the number of the available observations, where the number of its columns will

be equivalent to the number of the involved unknowns. As a result, the coefficient matrix A can be given as:

$$A_{n*u} = \begin{bmatrix} \frac{\partial f_1}{\partial X} & \frac{\partial f_1}{\partial Y} & \frac{\partial f_1}{\partial Z} & \frac{\partial f_1}{\partial t} & \frac{\partial f_1}{\partial N1_{sat\#1}} & \frac{\partial f_1}{\partial N2_{sat\#1}} & -- & -- & \frac{\partial f_1}{\partial N1_{sat\#n}} & \frac{\partial f_1}{\partial N2_{sat\#n}} \\ \frac{\partial f_2}{\partial X} & \frac{\partial f_2}{\partial Y} & \frac{\partial f_2}{\partial Z} & \frac{\partial f_2}{\partial t} & \frac{\partial f_2}{\partial N1_{sat\#1}} & \frac{\partial f_2}{\partial N2_{sat\#1}} & -- & -- & \frac{\partial f_2}{\partial N1_{sat\#n}} & \frac{\partial f_2}{\partial N2_{sat\#n}} \\ \frac{\partial f_3}{\partial X} & \frac{\partial f_3}{\partial Y} & \frac{\partial f_3}{\partial Z} & \frac{\partial f_3}{\partial t} & \frac{\partial f_3}{\partial N1_{sat\#1}} & \frac{\partial f_3}{\partial N2_{sat\#1}} & -- & -- & \frac{\partial f_3}{\partial N1_{sat\#n}} & \frac{\partial f_3}{\partial N2_{sat\#n}} \\ -- & -- & -- & -- & -- & -- & -- & -- & -- & -- \\ -- & -- & -- & -- & -- & -- & -- & -- & -- & -- \\ -- & -- & -- & -- & -- & -- & -- & -- & -- & -- \\ \frac{\partial f_{u-2}}{\partial X} & \frac{\partial f_{u-2}}{\partial Y} & \frac{\partial f_{u-2}}{\partial Z} & \frac{\partial f_{u-2}}{\partial t} & -- & -- & -- & -- & \frac{\partial f_{u-2}}{\partial N1_{sat\#n}} & \frac{\partial f_{u-2}}{\partial N2_{sat\#n}} \\ \frac{\partial f_{u-1}}{\partial X} & \frac{\partial f_{u-1}}{\partial Y} & \frac{\partial f_{u-1}}{\partial Z} & \frac{\partial f_{u-1}}{\partial t} & -- & -- & -- & -- & \frac{\partial f_{u-1}}{\partial N1_{sat\#n}} & \frac{\partial f_{u-1}}{\partial N2_{sat\#n}} \\ \frac{\partial f_u}{\partial X} & \frac{\partial f_u}{\partial Y} & \frac{\partial f_u}{\partial Z} & \frac{\partial f_u}{\partial t} & -- & -- & -- & -- & \frac{\partial f_u}{\partial N1_{sat\#n}} & \frac{\partial f_u}{\partial N2_{sat\#n}} \end{bmatrix}$$

(2-91)

Where:

$f_1 \rightarrow$ denoted for the first observation type P_{L1}.

$f_2 \rightarrow$ denoted for the first observation type P_{L2}.

$f_3 \rightarrow$ denoted for the first observation type Φ_{IF}.

$$\frac{\partial f_i}{\partial X} = \frac{x - Xs}{\rho}, \quad \frac{\partial f_i}{\partial X} = \frac{x - Xs}{\rho}, \quad \frac{\partial f_i}{\partial X} = \frac{x - Xs}{\rho}, \quad i = 1,2,3 \qquad \text{(2-92)}$$

$$\rho = \sqrt{(Xs - x)^2 + (Ys - y)^2 + (Zs - z)^2}$$

(2-93)

x,y,z are unknown receiver coordinates, Xs, Ys, Zs are the satellite known coordinates.

$$\frac{\partial f_i}{\partial t} = C \qquad \text{,C is the speed of light.} \qquad \text{(2-94)}$$

$$\frac{\partial f_i}{\partial N1} = 0 \; or \; 1 \qquad \frac{\partial f_i}{\partial N2} = 0 \; or \; 1 \qquad \text{(2-95)}$$

he coefficient matrix A is also known as the first design matrix. In addition to this design matrix, there are two other design matrices that must be formed and used with the matrix A in the process of the parametric least-squares adjustment. These two matrices are called the weight matrix P and the misclosure vector W. The construction of the three design matrices A, P and W is considered the core of the application of the parametric least-squares adjustment process. The weight matrix can be written as:

$$P_{n*n} = \sigma^2 . \sum_{n*n} L^{-1} \qquad (2\text{-}96)$$

Where σ^2 is the apriori variance factor and ΣL is the variance-covariance matrix of the observations. The weight matrix can be rewritten as:

$$P_{n*n} = \begin{bmatrix} \sigma^2_{1_{sat\#1}} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \sigma^2_{2_{sat\#1}} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & \sigma^2_{3_{sat\#1}} & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & -- & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & -- & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & -- & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & \sigma^2_{1_{sat\#n}} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & \sigma^2_{2_{sat\#n}} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \sigma^2_{3_{sat\#1}} \end{bmatrix} \qquad (2\text{-}97)$$

Where $\sigma_1^2 = \sigma^2_{P_{P_{L1+\Phi L1}}}$, $\sigma_2^2 = \sigma^2_{P_{P_{L2+\Phi L2}}}$, $\sigma_3^2 = \sigma^2_{\Phi_{IF}}$

Finally, the misclosure vector W, can be formed as:

$$\underset{n*1}{W} = \underset{n*1}{L^0} - \underset{n*1}{L} \qquad (2\text{-}98)$$

$$\overline{X} = X^0 + \delta \qquad (2\text{-}100)$$

In which L^0 is the vector of the observations, using the approximate values of the unknowns X^0, and L is the vector of observations. After formation of the three design matrices A, P and W, the solution of the unknown correction vector δ can be obtained as:

$$\underset{u*1}{\delta} = -(A^t.P.A).(A^t.P.W) \qquad (2\text{-}99)$$

Finally, the best estimator of all the considered unknowns, which is usually denoted as $\overline{X}$ can be obtained by adding the resulted correction vector δ to the pre-chosen approximate values of the unknowns X^0. Hence, the adjusted values of the unknowns can be expressed as:

Chapter three

Introduction to Geodetic Datums

Geodesy is the science of measuring the shape and size of the Earth and precisely locating points on its surface. As our society and economy becomes increasingly dependent on complex technologies and the management of the space we live in, the need for precise positioning and consistent, reliable spatial data has intensified. As we move to a world where new technologies allow us to rapidly determine the accurate position of features and points, we are developing the concept of everything „geodetic‟. That is the development of a seamless geodetic cadaster and all spatial datasets in terms of a common geodetic system.

For many countries subject to the effects of ground movements due to events such as earthquakes, volcanic activity or plate tectonics, the ability to survey and record these movements to maintain accuracy of the geodetic system is an important task. A country‟s geodetic system provides the network of permanent ground reference points and the associated intellectual and positional data that enables it to ensure all data concerning land, resources, and location is managed in a systematic and orderly manner.

Fundamental to any geodetic system is the spatial reference frame upon which it is based. Historically these were locally or regionally based, but as we have transitioned to the use of globally based satellite positioning systems our reference frames have become much more global in nature. A spatial reference frame allows a location to be unambiguously identified through a set of coordinates (usually latitude and longitude or northing and easting).

3.1 Introduction to Geodetic Datums

A geodetic datum is a curved reference surface that is used to express the positions of features consistently. Geodetic datums are usually classified into two categories: local and geodetic.

A Local Geodetic Datum is a datum which best approximates the size and shape of a particular part of the Earth's sea-level surface (Figure 3.1). It is defined by specifying a reference ellipsoid, the position (latitude and longitude) of an initial station and an azimuth from that station. Invariably, the center of its ellipsoid will not coincide with the Earth's center of mass. Until very recently, most national geodetic systems were based on local geodetic datums [13].

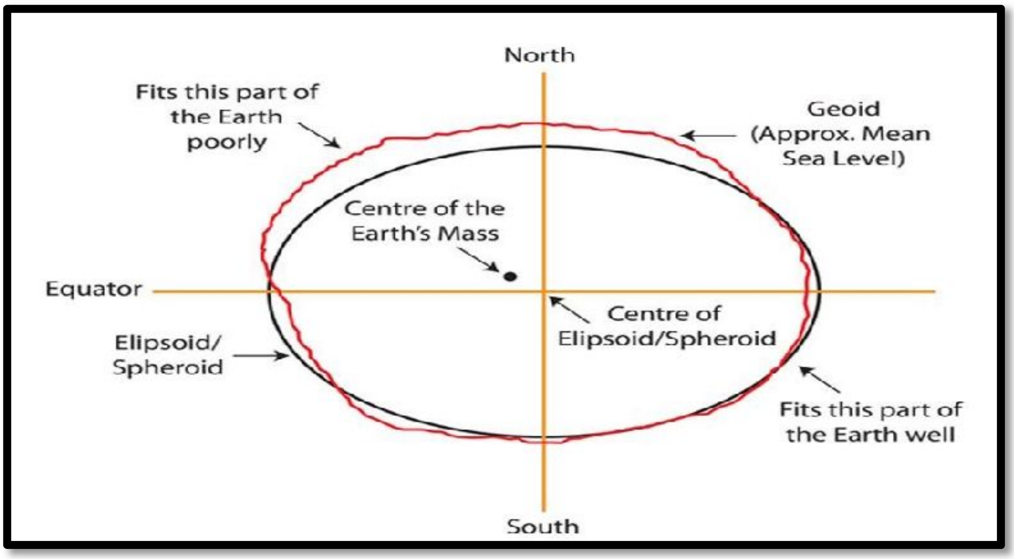

Figure (3.1.): Local datum with best fit ellipsoid

A Geocentric Datum is a datum which best approximates the size and shape of the Earth as a whole. The center of its ellipsoid coincides with the Earth's center of mass (Fig. 3.2). Geocentric datums do not seek to be a good approximation to any single part of the Earth but on average they are a good fit. Global Navigation Satellite Systems (GNSS) utilize geocentric datums to express their positions because of their global extent. Multiple GNSS are now fully operational or being developed such as GPS, GLONASS, GALILEO, and BEIDOU and each uses a slightly different geocentric datum. The World Geodetic System 1984 (used by GPS) is an example of a geocentric datum [13].

Mean sea level is widely used as the reference surface for the measurement of height. The contours on a map will usually show height above mean sea level. However, heights in terms of a geodetic datum will be in relation to an ellipsoid [13].

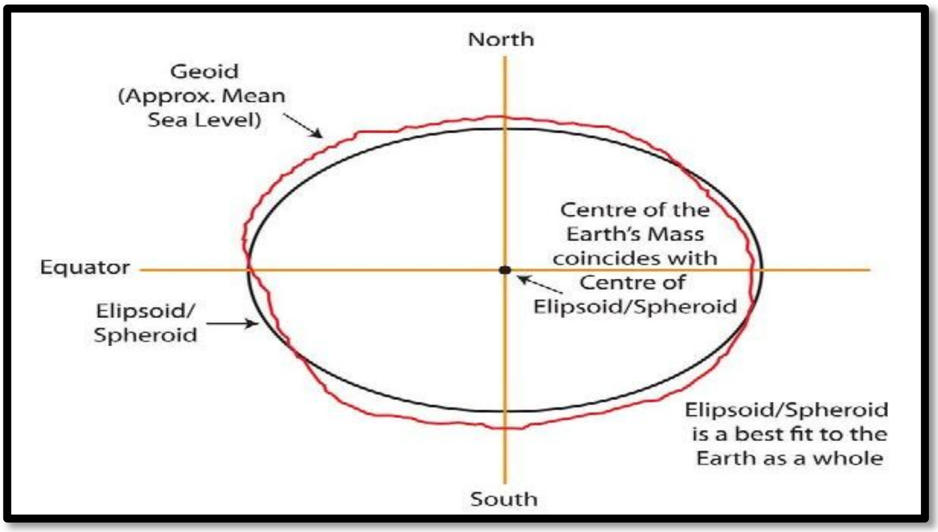

Figure (3.2.): Geocentric datum with ellipsoid that is a best fit to the world

3.2 Modern Geodesy and ITRS/ITRF

The primary mission of Modern Geodesy is the definition and maintenance of precise geometric and gravimetric reference frames and models, and the provision of high accuracy positioning techniques for users in order to connect to these frames. The International Association of Geodesy (IAG) has established services for all the major satellite geodesy techniques that are vital to the definition of the terrestrial and celestial reference systems. These reference systems are the foundation for all operational geodetic applications associated with mapping and charting, navigation, spatial data acquisition and management, as well as support for the geosciences.

The International Celestial Reference System (ICRS) forms the basis for describing celestial coordinates, and the International Terrestrial Reference System (ITRS) is the foundation for the definition of terrestrial coordinates to the highest possible accuracy. The definitions of these systems include the orientation and origin of their axes, scale, physical constants and models used in their realization, e.g., the size, shape and orientation of the reference ellipsoid that approximates the geoid and the Earth's gravity field model. The coordinate transformation between the ICRS and ITRS is described by a sequence of rotations that account for variations in the orientation of the Earth's rotation axis and its rotational speed [14].

3.2.1 International Reference Systems, Frame and Datum maintenance

In space geodetic positioning, where the observation techniques provide absolute positions with respect to a consistent terrestrial reference frame, the corresponding precise definition and realization of terrestrial and inertial reference systems is of fundamental importance. Thanks to significant improvements in receiver technology, to extension and densification of the global tracking network along with more accurate determination of positions and velocities of the tracking stations and to dramatically improved satellite orbits, GPS is today approaching one ppb precision for longer baselines and it can be considered to be a global geodetic positioning system providing nearly instantaneous three-dimensional position at the 1 - 2 cm accuracy level. With respect to this the reference system is one of the primary limiting error sources. One of the fundamental goals of geodesy is to precisely define positions of points on the surface of the Earth, so it is necessary to establish a well-defined geodetic datum for geodetic measurements and positioning computations. Recently, a set of the coordinates established by using GPS and referred to an international terrestrial reference frame could be used as a three-dimensional geocentric reference system for a country [15].

3.2.2 Terrestrial Reference Systems and Frames

Reference System and Reference Frame are different concepts. The first one is understood as “a theoretical definition”, including models and standards for its implementation. The second one is its “practical implementation” through observations and a set of reference coordinates (set of fundamental stars –for a Celestial Reference Frame– or fiducial stations –for a Terrestrial Reference Frame described by a catalogue of precise positions and motions (if measurable) at a specific epoch. A reference system is the complete conceptual definition of how a coordinate system is formed. It defines the origin and the orientation of fundamental planes or axes of the system. Satellite coordinates and user receivers must be expressed in a well-defined reference system. Thence, an accurate definition and determination of such systems is essential to assure a precise positioning in GNSS. Two of the main reference systems used in satellite navigation are introduced bellow: The Conventional Celestial Reference System (CRS) (also named Conventional Inertial System, CIS) and the Conventional Terrestrial Reference System (also named Coordinated Terrestrial System, CTS)

In satellite geodesy two fundamental systems are required: a space- fixed, conventional inertial reference system (CRS) for the description of satellite motion, and an Earth-fixed, conventional terrestrial reference system (CTS) for the positions of the observation stations and for the description of results from satellite geodesy [4].

Conventional Celestial Reference System (CRS)

The concept of a celestial sphere and the definition of some basic planes are necessary for establishing a reference system for earth center inertial (ECI) coordinates. The celestial sphere is an imaginary sphere of infinite radius whose center coincides with the center of mass of the earth. The celestial poles and celestial equator are, respectively ,projections of the earth's north and south astronomic poles and astronomic equator onto the celestial sphere. The vernal and autumnal equinoxes are the points where the celestial equator intersects the ecliptic.

This is a inertial reference system. It has its origin at the earth's center of mass. X-axis points in the direction of the mean equinox at J2000.0 epoch, Z-axis is orthogonal to the plane defined by the mean equator at J2000.0 epoch (fundamental plane) and Y-axis is orthogonal to the former ones figure (3.3), so the system is directly (right handed) oriented. The practical implementation is called (conventional) Celestial Reference Frame (CRF) and it is determined from a set of precise coordinates of extragalactic radio sources (i.e., it is fixed with respect to distant objects of the universe). The mean equator and equinox J2000.0 were defined by International Astronomical Union (IAU) agreements in 1976, with 1980 nutation series [13].

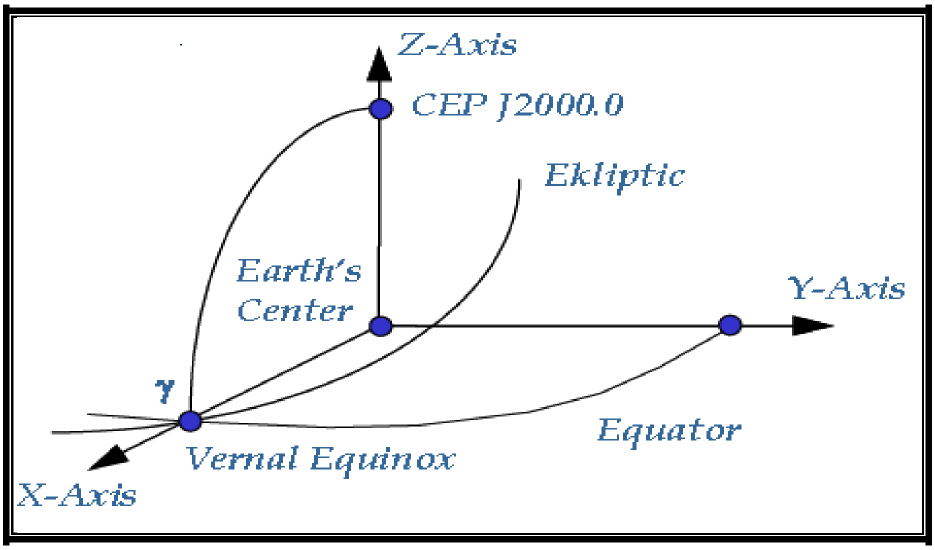

Figure (3.3.): Conventional Celestial System (CRS)

Conventional Terrestrial Reference System (CTS)

This is a reference system co-rotating with the earth in its diurnal rotation, also called Earth-Centered, Earth-Fixed (ECEF). Its definition involves a mathematical model for a physical earth in which point positions are expressed and have small temporal variations due to geophysical effects (plate motion, earth tides, etc.). The CTS has its origin in the earth's center of mass. Z-axis is identical to the direction of the earth's rotation axis defined by the Conventional Terrestrial Pole (CTP), X-axis is defined as the intersection of the orthogonal plane to Z-axis (fundamental plane) and Greenwich mean meridian, and Y-axis is orthogonal to both of them, making the system directly oriented figure (3.4). In solving practical problems of navigation, geodesy, geodynamics, geophysics and other geosciences it is necessary to have coordinate systems firmly connected with the earth body. Therefore, it is necessary to define a terrestrial geocentric system which would make it possible to solve global positioning problems at the highest accuracy level. The Conventional Terrestrial Reference System adopted for either the analysis of individual data sets by observation techniques [13].

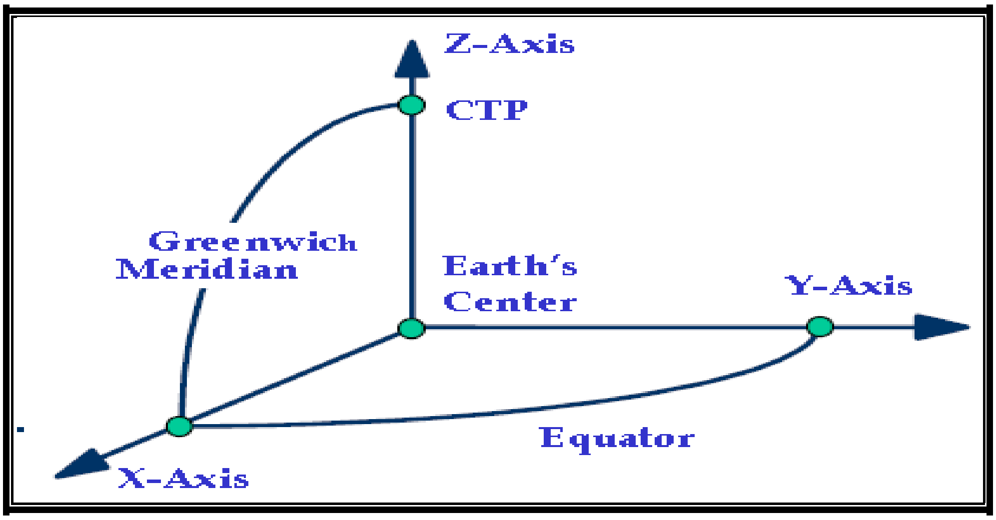

Figure (3.4.): Conventional Terrestrial System (CTS)

International Terrestrial Reference System (ITRS)

The ITRS represents the most precise global terrestrial reference system which is a source system for the realization of other world reference systems (e.g. WGS84) and of continental or regional reference systems (e.g. ETRS89 etc.). ITRS has its origin on the earth's center of mass (including the Ocean and atmosphere). Z axis pointing toward CTP known as IRP (IERS Reference Pole), X axis on the Earth equator and IRM (IERS Reference Meridian passing Greenwich) and Y-axis is orthogonal to X and Z axes figure (3.5). table (3.1) describes the differences between reference systems. [13]

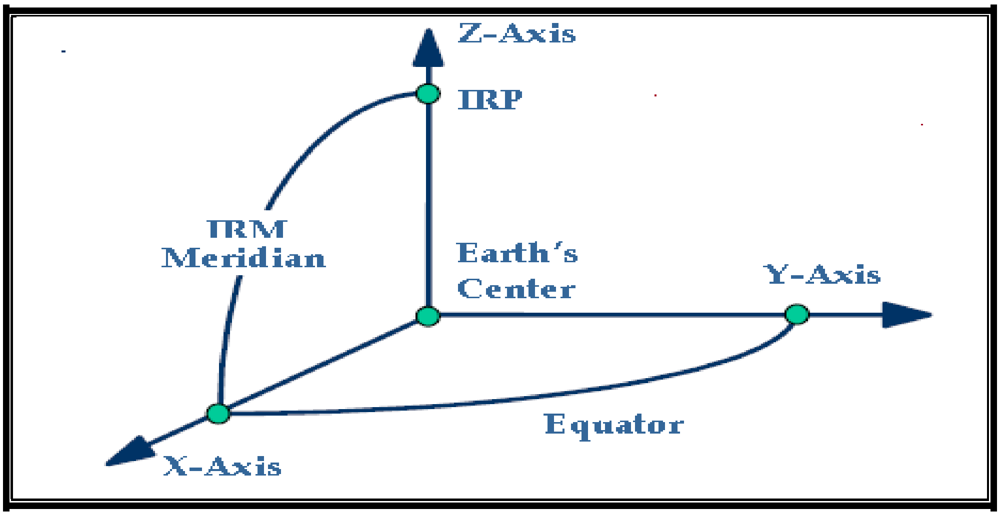

Figure (3.5.): International Terrestrial Reference System (ITRS)

Table (3.1.) Differences between reference systems

Parametes	Reference System		
	CTS	**ITRS**	**CIS**
Origin	Mass Center of the Earth	Mass Center of the Earth (including the Ocean an atmosphere)	Mass Center of the Earth
Cordite System	Earth –F0ixed	Earth -Fixed	Space-Fixed
X-axis	On the Earth equator And Greenwich meridian plane	On the Earth equator and IRES Reference meridian passing Greenwich	On the Earth equator and pointing to the Vernal Equinox of J2000.0 epoch
Y-axis	Orthogonal to X and Z axis	Orthogonal to X and Z axis	Orthogonal to X and Z axis
Z-axis	Pointing Toward CTP (Conventional Terrestrial Pole)	Pointing Toward CTP (IERS Reference Pole)	Pointing Toward CEP (Conventional Ephemeris pole of J2000.0 epoch
Example	WGS 84	ITRF	-------------

[16]

3.2.3 International Terrestrial Reference Frame (ITRF)

The realization of International Terrestrial Reference Systemis named (conventional) Terrestrial Reference Frame (TRF) and it is carried out through the coordinates of a set of points on the earth serving as reference points .A conventional TRF is defined as a set of physical points with precisely determined coordinates in a specific coordinate system that is the realization of an ideal TRS [17] An example of TRF is the International Terrestrial Reference Frame (ITRF) introduced by the International Earth Rotation and Reference Systems Service (IERS), which is updated every year (ITRF98, ITRF99, etc.). Other terrestrial reference frames are the World Geodetic System 84 (WGS84), which is applied for GPS .The types of different TRF can be described by the following [17]:

Global Reference Frames

A global reference frame is typically the primary basis for the definition of a coordinate system used in applied geodesy. Examples include the International Terrestrial Reference Frame (ITRF) and the World Geodetic System 1984 (WGS 84). These frames are geocentric in nature, having the geocentric (the center of mass of the Earth) as the origin and orthogonal axes aligned with pole, equator and Greenwich meridian according to IERS conventions. The ITRF is realized by the coordinates and site velocities of a network of global stations and forms the basis for modern regional and national reference frames or geodetic datums. The most recent realizations of ITRF have positional uncertainties of contributing stations in the order of millimeters. ITRF station velocities are described with respect to a no-net-rotation (NNR) condition where the angular momenta of all of the global tectonic plates sum to zero.

Regional Reference Frames

Regional reference frames are denser networks of geodetic stations covering continental areas. Examples include the European Terrestrial Reference Frame (EUREF), North American Datum 1983 (NAD83), African Reference Frame (AFREF), Sistema de Referencia Geocentrico para las America (SIRGAS) and the Asia-Pacific Reference Frame (APREF). As with ITRF, regional reference frames are defined by the coordinates and site velocities of contributing stations. The key difference with some regional reference frames (e.g. EUREF and NAD83) and ITRF is that the site velocities may be with respect to the dominant tectonic plate encompassed by the frame and not a NNR condition. This approach minimizes site velocities. Regional frames not constrained by the motion of a single tectonic plate are closely aligned with ITRF.

National Reference Frames

Modern national reference frames are typically a static realization of ITRF or a regional reference frame. In most countries the coordinates of a national reference frame (or geodetic datum) form the basis for all surveying, positioning and mapping within national borders. Because surveying/ GIS software and spatial data are not generally designed to deal with continuously changing coordinates, the epoch for national datums is fixed and the coordinates are considered to be invariant with time.

The key-element of the ITRF combinations is the availability of co-location sites where two or more space geodesy instruments are operating and where differential coordinates(local ties) between the measuring reference points of these instruments are determined. While any individual space geodesy technique Very Long Baseline Interferometry **(VLBI**), Satellite Laser Ranging **(SLR),**Global Positioning System **(GPS**) and Doppler Orbitography Radio positioning Integrated by Satellite (DORIS) [17] .Global Navigation Satellite Systems (GNSS) including Galileo in the future) is able to provide all the necessary information of the datum definition of the Terrestrial Reference Frame (TRF) (origin ,scale, orientation), combination of reference frames provided by independent techniques has long been the standard method to implement global terrestrial reference frames .The latest realization of the international terrestrial reference frame during this study is known as ITRF2005, but now is ITRF2008 and is derived from several spaced-based, geodetic techniques such as Global Navigation Satellite Systems (GNSS).

3.3 World Geodetic System (WGS84)

WGS84 or the World Geodetic System 1984 is the geodetic reference system used by the GNSS - “GPS”. WGS84 was developed for the United States Defense Mapping Agency (DMA), now called NGA (National Geospatial -Intelligence Agency). WGS84 is the default "native" system used by the Global Positioning System (GPS) and commercial GPS receivers**.** Although the nameWGS84 has remained the same, it has been enhanced on several occasions to a point where it is now very closely aligned to ITRF and referenced as WGS 84 (G1150). It was used as the reference frame for broadcast GPS Ephemerides (orbits) beginning January 23, 1987. At 0000 GMT It was redefined again and was more closely aligned with International Earth Rotation Service (IERS) Terrestrial Reference Frame (ITRF)**.**The origin of the WGS 84 Coordinate System is the center of mass of the earth ;Z-axis is parallel to the direction of the CTP for polar motion, as defined by the Bureau International de ĹHeure (BIH) on the basis of the coordinates adopted for the BIH stations; the X-axis is the intersection of the WGS 84 reference meridian plane and the plane of the CTP's equator, the reference meridian being parallel to the Zero Meridian defined by the BIH on the basis of the coordinates adopted for the BIH stations; and, the Y-axis, measured in the plane of the above equator, 90" east of the X-axis, completes a right-handed ,earth-fixed, orthogonal coordinate system figure (3.6). The use of global kinematic reference frames such as ITRF and WGS84 in positioning, navigation and mapping is now widespread. Positions within these reference frames can be acquired with a precision of between a few millimeters and several meters depending upon the choice of GNSS receiver, differential service or processing method.

A major dilemma arises when these positioning technologies are used to reposition fixed locations on the Earth's surface (e.g. land surveying, airborne laser scanning, deformation monitoring, precision agriculture and automated mining). Coordinates of fixed locations within a kinematic reference frame (or "dynamic datum") change by up to 100 mm/yr due to plate tectonics. Where large earthquakes occur, coordinate changes of up to several meters in magnitude are possible. Unmodeled deformation is undesirable if surveys referenced to different measurement epochs of a kinematic datum are to be integrated or correlated [16].

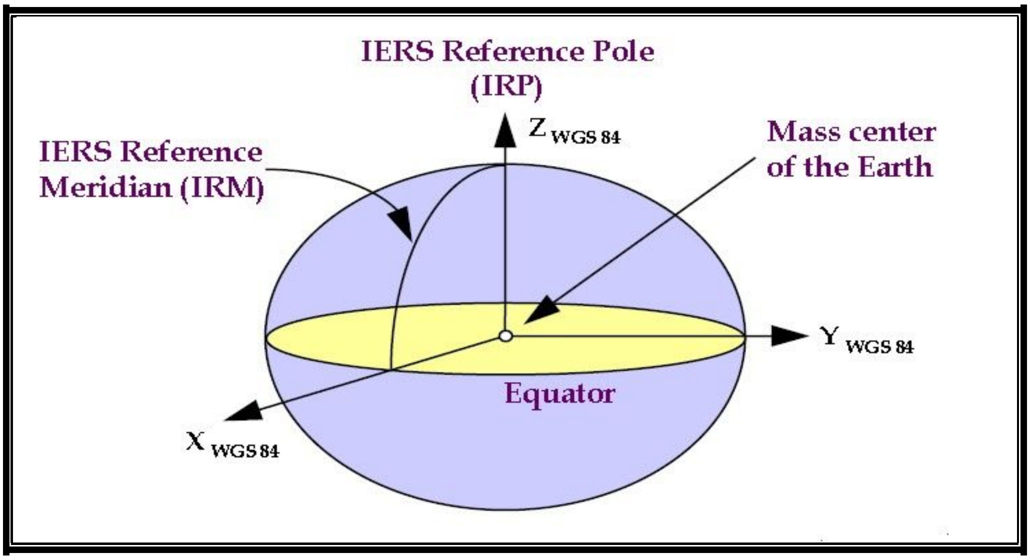

Figure 10 (3.6.): World Geodetic System 1984 (WGS84)

3.4 The Datum Problem

In the classical sense, a geodetic datum is a reference surface, generally an ellipsoid of revolution of adopted size and shape, with origin, orientation, and scale defined by a geocentric terrestrial frame. Once an ellipsoid is selected, coordinates of a point in space can be given in Cartesian or geodetic (curvilinear) coordinates (geodetic longitude, latitude, and ellipsoid height). Geodetic coordinates are preferred in cartographic and mapping applications. Furthermore, the classical concept of geodetic datum implies that a datum's coordinates are fixed and do not change with time except for the effect of local tectonic motion (episodic motions, land subsidence, volcanic activity, etc.). Thus, the coordinate frame of a geodetic datum should be somewhat attached to the plate and move with it in such a way that the coordinates of the points will not change as a consequence of plate rotation.

However, in actuality, the reverse process is implemented; that is, the coordinate frame is fixed to the Earth's mantle while the plates are rotated to their original position at epoch $D\ t$ (the datum epoch). This is achieved by applying the same type of correction at every point. The magnitude of this correction is determined through the angular velocity matrix associated with the continental plate where the points are located. In essence, all points are moved back to their location at epoch $D\ t$ on the frame ITRF which, in our example, is assumed to be the adopted datum frame and, by definition, remains fixed. In other words, the plate and the points on it are assumed frozen in space at the epoch when the datum frame was defined; all coordinates determined at epoch t should be taken back to epoch $D\ t$, the datum epoch [13].

3.4.1 Limitations of Static Geodetic Datums

Two types of geodetic datum can be defined namely a static and kinematic geodetic datum. A static datum is thought of as a traditional geodetic datum where all sites are assumed to have coordinates which are fixed or unchanging with time. This is an incorrect assumption since the surface of the earth is constantly changing because of tectonic motion .Static datum does not incorporate the effects of plate tectonics and deformation events. Coordinates of static datum are fixed at a reference epoch and slowly go out of the date, need to change periodically which is disruptive.

The concept of dynamic (or kinematic) datum has been introduced to represent a coordinate datum where the coordinates of sites change as a function of time [18] Dynamic datum incorporates a deformation model to manage changes (plate tectonics and deformation events). Coordinates change continuously and can be confusing and difficult to manage.

Countries and regions located on relatively stable tectonic landmasses such as Egypt, South Africa, Brazil, Eastern USA, Germany and the UK have geodetic datum that do not deform significantly as a function of time. Typically, baselines measured between any stable geodetic monuments in these countries change by less than a few centimeters on decadal time scales. This stability has supported the adoption of a static geodetic datum for such regions.

The major limitation with a static geodetic datum arises from the ongoing divergence between ITRF and the fixed coordinates of a static geodetic network due to rigid plate motion. While the network may not necessarily be deforming internally to any significant degree, the lithospheric plate on which the network sits is moving as a rigid body over the Earth's as the no spherical mantle. The impact of this deformation is noticeable where precision GNSS techniques are used to compute ITRF coordinates.

Another limitation of a static datum arises from the processing of long GNSS baselines. If the static coordinates of a reference station are held fixed, rigid plate rotation of a long baseline will degrade the precision of the point computation as a function of time [19].

Global PPP systems and post-processing services such as Omni Star, and NRCan, provide instantaneous ITRF coordinates which will be invariably misaligned from any static realization of ITRF, unless the position is also explicitly stated in a static datum (e.g. NAD83 for OPUS and NRCan; GDA94 for AUSPOS). Using a simplified plate based transformation model can enable the ITRF solution to be related to a fixed epoch. If such a transformation strategy could be implemented, users could either choose a static epoch, or use a database of existing datum with defined reference epochs and origin translations. A polygon file for each rigid plate can define the extents of rigid plates and deforming zones, so that the correct parameter set and deformation model can be implemented depending upon the user's position. Alternatively, datum specific online processing services could be developed, so that users are spared the need to perform additional transformations [20] **.** So, datum's can either become fully kinematic (dynamic), or semi-kinematic. A deformation model can be adopted to enable ITRF positions to be transformed into a static or semi-kinematic system at the moment of position acquisition so that users do not see coordinate changes due to global plate motions.

3.4.2 Limitations of Kinematic Geodetic Datum

GNSS devices which use ITRF or closely aligned systems position users in agreement with the underlying kinematic frame, however, in practice there are a number of very significant drawbacks to a kinematic datum. Surveys undertaken at different epochs cannot be combined or integrated unless a deformation model is applied rigorously, or is embedded within the data, and the data are correctly time-tagged. Three-dimensional data sets acquired by laser scanning techniques (e.g. airborne or terrestrial laser scanning) are often several terabytes in size and comparison of point clouds offset by a different realization within a kinematic system imposes an increased computational workload and potential for error. High-precision automated GNSS techniques are rapidly being adopted in the agricultural, mining and transport sectors (e.g. aviation, lane control and shipping). Unless spatial models of farm machinery tracks, mine infrastructure, berths and runways also move several centimeters a year to maintain alignment with a kinematic system, the limitations of a kinematic datum soon become apparent as kinematic and ground-fixed coordinates become misaligned [20] .

3.4.3 Benefits of a Semi-Kinematic Datum

Semi –Kinematic datum incorporates a deformation model to manage changes (plate tectonics and deformation events). Coordinates fixed at a reference epoch, so the change to coordinates is minimized. Many countries and regions which straddle major plate boundaries have adopted a semi-kinematic (or semi-dynamic) geodetic datum in order to prevent degradation of the datum as a function of time due to ongoing crustal deformation that is occurring within the country .The two major drivers for the adoption of a semi-kinematic datum in these countries have been: (1) the widespread adoption of precision GNSS techniques for surveying and positioning and, (2) the need to maintain consistency of coordinates to support combination and integration of spatial information acquired at different epochs [21].

High precision GNSS positioning and navigation is very rapidly highlighting the disparity between global kinematic reference frames such as ITRF and WGS84, and traditional static geodetic datum. The disparity is brought about by the increasingly widespread use of PPP and the sensitivity of these techniques to deformation of the Earth due to plate tectonics. In order for precision GNSS techniques to continue to deliver temporally stable coordinates within a localized reference frame [20].

Also, PPP, global RTK and GNSS post-processing services are now used extensively to provide realizations of ITRF and WGS84 globally with a precision of a few centimeters. Unless these instantaneous realizations are transformed to a static or semi-kinematic datum using a suitable kinematic transformation model, repeat surveys using these techniques will result in datum divergence as a function of time arising from the effects of unmodeled tectonic plate motion. Africa has a very sparse CORS infrastructure, and this limitation supports the use of PPP and related techniques. At present, there is no kinematic transformation applied through these services to maintain consistency of coordinate solutions, which account for plate motion. The simplified transformation method can be easily coded into GNSS algorithms to enable a reference epoch to be chosen by the user. All of these services initially compute positions in the latest realization of the International Terrestrial Reference Frame (ITRF) or closely aligned WGS84 (G1150) (NGA, 2004).

3.5 Kinematic Transformation Parameters Using Rigid Plate Rotation Model

Transformations from kinematic ITRF to a static datum are conventionally done by either using the site velocity (measured directly or computed from a plate motion model) to compute the displacement between the reference and current epochs or by a conformal transformation augmented with time dependent parameters to account for rigid plate motion Fig (3.7) e.g., Geoscience Australia's 14-parameter model [19] **.** The precision achievable is often several millimeters on a decadal time scale within any rigid plate.

Most larger tectonic plates (e.g. the Pacific and Australian Plates) move as a rigid body with almost insignificant intraplate deformation away from the plate boundaries (with the exception of spasmodic and rare intraplate earthquakes) [22] . Plate movement is conventionally defined by a rotation rate about an Euler Pole.

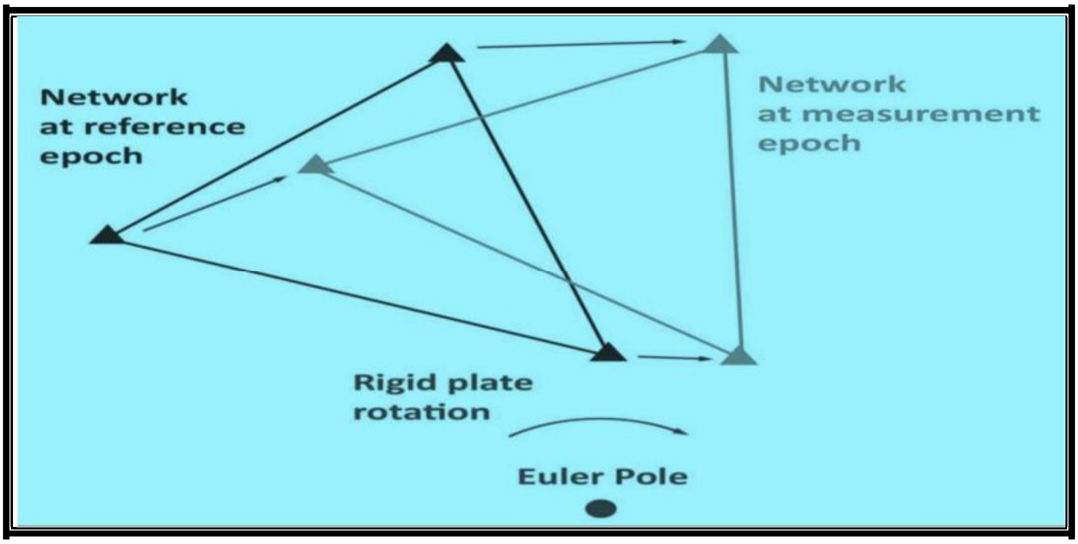

Figure (3.7.): Rigid Plate Rotation

The Earth's surface is comprised of a number of tectonic plates .These plates collide, rift apart, or slip past adjoining plates along the plate margins at rates of up to several centimeters a year. Major earthquakes and volcanic activity predominantly occur within these plate boundary zones. In general, tectonic plates are internally rigid and stable away from the plate boundaries. Baselines measured between any two geologically and structurally stable geodetic stations located on a rigid plate are unlikely to change by more than a few mm/yr. Conversely, within plate boundary zones and regions of diffuse deformation (e.g. Tibetan plateau and the Eastern Mediterranean), baseline changes become significant and highly variable depending upon the strain regime prevalent within the deformation zone.

Approximately 94% of the Earth's surface lies on rigid tectonic plates where localized deformation rarely exceeds more than a few mm/yr [20]. Rigid plates consist of segments of the Earth's crust rotating over the mantle. Rigid plate motion can be parameterized by three parameters, either by definition of the Euler pole of rotation, or the rotation of the Cartesian axes with respect to the inertial Earth frame .ITRF or WGS84 coordinates can be transformed to a regional static geocentric datum by using a four parameter model derived from absolute rigid plate kinematic models .Within tectonically stable areas.

A site velocity in Cartesian format (X.,Y.,Z.)(in meters) can be computed for any given location (*X, Y, Z* in meters) on a rigid plate defined by (Ω*X*, Ω*Y*, Ω*Z* in radians per million years) using:

$$\begin{bmatrix} X^{\cdot} \\ Y^{\cdot} \\ Z^{\cdot} \end{bmatrix} = \begin{bmatrix} \Omega_Y Z - \Omega_Z Y \\ \Omega_Z X - \Omega_X Z \\ \Omega_X Y - \Omega_Y X \end{bmatrix} .1E-6 \qquad (1.3)$$

By introducing a reference epoch and an epoch of measurement *t* (epochs in decimal years), the ITRF coordinates of any point on a rigid plate at a reference epoch (, , in meters) can be computed from the coordinates at epoch *t* (, , in meters) using :

$$\begin{bmatrix} X_0 \\ Y_0 \\ Z_0 \end{bmatrix} = \left[\begin{bmatrix} X_t \\ Y_t \\ Z_t \end{bmatrix} + \begin{bmatrix} \Omega_Y Z_t - \Omega_Z Y_t \\ \Omega_Z X_t - \Omega_X Z_t \\ \Omega_X Y_t - \Omega_Y X_t \end{bmatrix} .(t_0 - t)1E-6 \right] \qquad (2.3)$$

The above equation can also be used to realize a static geocentric datum aligned with ITRF at a specific reference epoch. Instantaneous ITRF positions measured at different locations and at different epochs on the same rigid plate can be related to the static datum at the reference epoch by using the same parameters (, , ,). In instances where a geocentric datum is offset from ITRF (for example, a datum aligned with an earlier realization of ITRF or WGS84), three additional parameters (, ,) can be added to the transformation model to account for the translation of the ITRF origin from the datum at the reference epoch using:

$$\begin{bmatrix} X_0 \\ Y_0 \\ Z_0 \end{bmatrix} = \begin{bmatrix} T_X \\ T_Y \\ T_Z \end{bmatrix} + \left[\begin{bmatrix} X_t \\ Y_t \\ Z_t \end{bmatrix} + \begin{bmatrix} \Omega_Y Z_t - \Omega_Z Y_t \\ \Omega_Z X_t - \Omega_X Z_t \\ \Omega_X Y_t - \Omega_Y X_t \end{bmatrix} .(t_0 - t)1E-6 \right] \qquad (2.3)$$

The above equations are accurate within rigid plate zones, however within deforming zones additional parameters derived from deformation models (e.g. Finite Element Model and Fault Locking models) are required in order to maintain consistency between different epochs [19] . In addition, co-seismic and post seismic terms need to be added. This topic and its impact on the Palestine_1923 Grid Network will explain in detail in the next chapter.

Chapter Four

Geodetic Control Networks Using GNSS

A national horizontal control network is a fundamental construction project. We therefore need to proceed from the real situations of a nation, properly handle the relationship between quality, quantity, time, and expenditure according to the or y and the real experience of network establishment, and work out specific principles as the basis for designing and establishing the geodetic control networks.

4.1 Network Establishment and Control Based on Hierarchical Orders

A national horizontal control network can be established based either on a single order or several different orders. The single-order control network can serve directly as the basis for mapping control and is usually established in countries with smaller territories to ensure more homogeneous accuracy throughout the network and facilitate adjustment computations. Countries with vast territories often adopt the method of establishing networks from higher to lower orders.

They usually first build a nationwide primary control network with higher accuracy and lower density as a consistent control framework, and then continue to densify the control network in a piecemeal fashion according to the needs of different areas. The side lengths of control networks become shorter and the accuracies get lower as the order changes from higher to lower. Using such a method to establish other triangulation networks successively in different areas within one consistent coordinate system can not only satisfy the desired accuracy but also achieve the effective results at a faster pace and lower cost.

4.1.1 Sufficient Accuracy

Apart from being the control framework of the national unified coordinates, the first- and second-order networks, in the process of establishing the national horizontal control network, have to meet the requirements for mapping of the basic scale topographic maps and the development of modern technology, such as space technology, precise engineering, earthquake monitoring, and geodynamics, whereas the third- and fourth-order horizontal control networks are used chiefly for a higher-level control of the topographic mapping control points and to satisfy the needs of fundamental engineering construction. Control points of various orders, therefore, must cater for the actual demands. For example, the accuracy of the first- and second-order control points should meet the needs of a 1:50,000 scale topographic map, while that of the third- and fourth-order control points should meet the needs of topographic mapping at a scale of 1:10,000 [23].

4.1.2 Necessary Density

Density of the control points in the control network means that there is usually one single point every several square kilometers on average. It can also be expressed by the average side length of midpoints in the control network. The shorter the side length, the denser the geodetic points will be. The controlling area Q of each point is expressed by the average side length S, namely [23]:

$$Q = S^2, \quad S = \sqrt{Q}, \tag{4.1}$$

which is the relationship between the side length and the controlling area.

The density of the points is required to be different according to different mapping scales and methods. On average, three or four geodetic points are generally required to densify control points for each map sheet. For different engineering projects, however, the desired density of points will presumably be different and should be determined according to real situations.

4.2 Requirements for the Position of Control Points

The position of horizontal control points should satisfy the following requirements for either technical design or reconnaissance for control point selection:

1. The side lengths, angles, and graphical structures formed between Control Points should completely conform to the requirements in the corresponding technical standards.
2. The control points should be marked where the sites can be extended easily and lower-order points are conveniently densified.

3. The position should be selected where the survey mark can be well preserved over time and it will be safe and convenient to erect the monument and to observe it. Therefore, the position should be selected in high land with solid soil and a fine drainage system, and should be a suitable distance away from highways, railways, high-voltage wires, and other buildings.
4. The line of sight should go beyond or deviate from obstacles by a certain distance which, for first- and second-order, respectively, should be no less than 4 m and 2 m in mountainous areas while no less than 6 m and 4 m in plain areas.

4.2.1 Technical Design

- **Data Collection**

Data relevant to the survey areas should be collected before planning, including maps of various scales, aerial photo maps, traffic maps and meteorological information, existing results of geodetic points, natural and social geographical environments of the survey areas, transportation and material supplies, and so on. These data should be analyzed and studied as the basis and reference for the technical design.

- **Drawing Up Designs**

Drawing up designs is a key aspect in technical design that deserves careful consideration in order to facilitate site selection. Fieldwork will otherwise be difficult. Drawing up designs usually follows the steps and methods listed below [23]:

1. Splicing the 1/50,000 or 1/100,000 scale topographic maps of the survey area and marking the already established triangulation chains, GPS networks, traverse networks, and leveling lines on the map.
2. Extending outward from the points of known control in a pointwise manner according to the requirements for positions of control points while considering creating the best figure possible. The points are laid out from higher to lower orders, from points of known control to unknown control, and from the interior to exterior in a pointwise fashion.
3. Drawing up the leveling connection lines according to the density requirements for the zero-elevation surface provided in the corresponding technical standards;

Trying to utilize the old network points already existing and to propose the plans for a connection survey.

4. Ensuring indivisibility during the site selection. Several alternative plans should be drawn up for the uncertain positions of points or directions.

4.3 Erection of Survey Marks and Monument Setting

- **Erection of Survey Marks**

National triangulation points or traverse points are way apart from each other and invisible in general conditions; geodetic survey marks are therefore needed to show the specific position of the point as the target. Geodetic survey marks are usually classified into ordinary survey marks and tower structures. The former is only used as the target point with a height of 4.3 m and 6.3 m, while towers are chiefly used to elevate the instrument and position of the target point when two adjacent points are not indivisible [23].

- **Monument Setting**

The survey mark is a permanent mark of the control point position. Field observations referred to the center of the mark, and the plane coordinates and height of the point obtained will be the position of the survey mark center. If the mark is destroyed or displaced, the surveying results and position coordinates will be meaningless. Hence, when the monument is set, one should firmly abide by the principle of "quality first." Meanwhile, the monument should be stable to ensure permanence [23].

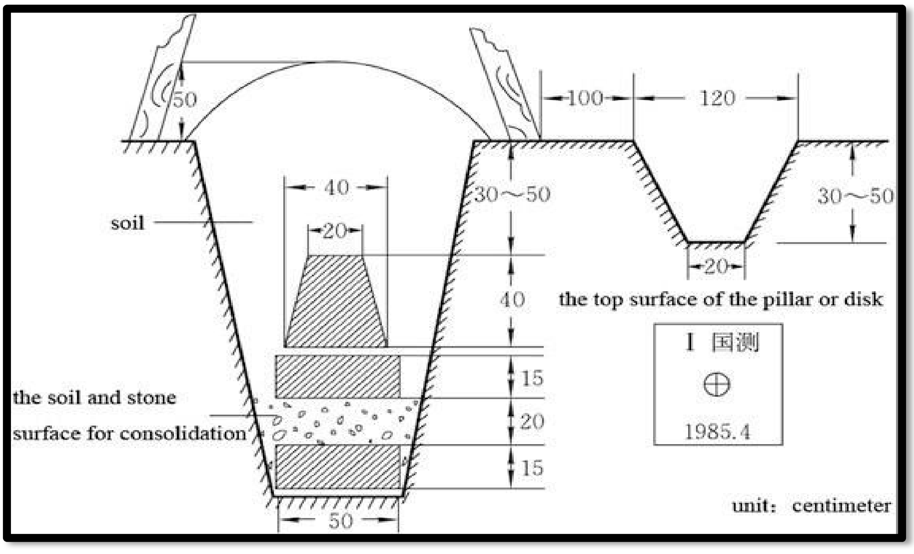

Figure (4.1.): Monumentation of the first- and second-order trig points

A monument can be classified as that of the first- and second-order triangulation (traverse) points or that of the third- and fourth-order triangulation (traverse) points. A monument is generally filled with concrete chiseled from granite, bluestone, or other hard stones with identical specifications. Monuments consist of disks and pillars, both with a mark sunken into the center of their top surfaces. The survey mark can be made of metal or vitreous enamel. There are many types of monuments, which are different in terms of the different orders and places of monumentation under the principle of ensuring their stability and permanence [23].

Generally, a monument of first- and second-order points is composed of pillars and upper and lower disks, as shown in figure (4.1), while that of third- and fourth order points is composed of pillars and one disk .

Completion of the technical design and erection of survey marks and monuments marks the position of each control point in the horizontal control network on the Earth's surface. However, extensive distance and angle measurements, as well as adjustment computations, still need to be made before the coordinates of the control points can be determined.

4.4 The Palestinian Geodetic Control Network

In Palestine, the measurement of the triangulation net and the control points was begun in 1921, In March 1921 the possibility was considered of measuring the baseline in the southern part; but it was measured in October along 4,730.6 meters in the Imara lands. At the same time, the possibility was investigated of measuring the check baseline of the system near Jenin. To calculate the topographic height of the triangulation points, the MSL was measured on the Gaza beach, and the altimetric measurements were connected to the Imara baseline by precise leveling. Towards the end of 1922 the net of fixed points was widely spreaded over most of the north, and the measurements were closed to the check line sited in the Haifa Bay, east of Acre, and not in the Jenin region as planned originally. In December 7th , the map of triangulation points showed that from the start there had been a clear intention to lay out the net only in the areas that were to be subjected to the cadastral survey in the future. No points were measured south; and so was the Huleh Valley, which at the time had still been excluded definitively within the territory of the Palestine Mandate [24].

In 1923 the major triangulation net of ninety-five fixed points was completed and marked in the field, but the measurements in the Galilee and Mount Carmel were not finished yet. In that year the gaps were closed, and fixed points were measured also in the mountain area north of Ramallah and the Jericho Valley, then the triangulation of Hebron was begun in March 1925, In April 1924, after the Huleh Valley became a part of Palestine, the northern border was finally demarcated to form the Huleh Salient (the 'Finger of Galilee'), and the Survey Department added five new points to the major triangulation net, and forty-three to the secondary net of third-order triangulation, to cover the 'newly acquired territory' by the survey [24].

Figure 13(4.2.): point 2M of the Palestine Major triangulation base line

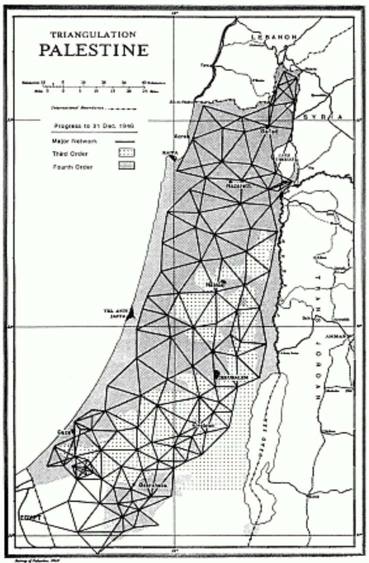

Figure (4.3.): The Major Triangulation system in Palestine

In this way the number of points in the major triangulation net reached 100, with point 100 sited, surprisingly in Syrian territory, above the old custom house at the Bridge of Jacob's Daughters across the Jordan. the Acre line was cancelled and it was decided to establish instead a check baseline south of the Sea of Galilee, near Samakh. This line was also measured a length of 2,901 meters, in the same plain where previously the baseline between points 1200 and 1201 had been measured in connection with the Beisan jiftlik. From the beginning, the two stations of the new line were marked 101M and 102M on the national major triangulation net. Later, however, they were given the numbers 66M and 67M of the points that had been planned but cancelled with the abandonment of the Acre line. In the closing survey that was conducted some time afterwards at the Samakh baseline, there proved to be a discrepancy between the computed trigonometric values and the actual measurement of the check line, and to straighten matters out the Egyptian Survey Department was called in to assist in conducting a professional check [24].

Categories of Triangulation survey

the survey began in May 1921. The first step was for the survey parties to lay out geodetic points throughout whole of the country, to measure their values, and to provide mathematical bases for the survey network. The geodetic points required for mapping are classified in three

categories: first Fixed points, or trigonometric stations, are determined by trigonometric methods must be in sight of each other for the surveying observations. These virtual lines form the sides of the triangles of the observation net. The data obtained are the position of the points in planimetric coordinates. The reduced level of the points is determined in relation to the mean sea level(MSL). second Spot heights are determined by accurate leveling and without the necessity to relate it to the trigonometric net. The topographic heights are measured in relation to the MSL along fixed runs in the field. there'd Gravimetric points, for the determination of the shape of the Earth. The net of fixed points therefore forms a basic national skeleton system into which link all the survey and mapping projects throughout the country. For these separate projects to link into the national net accurately and easily, the density of the measured points must be increased by splitting the major triangulation into secondary nets with triangles having shorter sides: these are the third- or fourth-order triangulation nets, and so on. Besides the measuring of triangulation nets, the number of triangulation points can be augmented so that in the detailed cadastral survey stage several points linked to the national reference net can be included in every map [24].

The geodetic projection for Palestine

The mapping of Palestine was influenced by the cartographic traditions in the colonies and by consideration of the available mathematical tables compiled and calculated beforehand in Britain and other countries. We do not know what prior considerations led the

British to select any particular geodetic projection for Palestine. The decision narrowed down between two projections: Gauss Conformal, known as Transverse Mercator Projection, and Cassini—Soldner, since these were accepted as convenient projections for both cadastral and topographic mapping. In 1922 the survey experts in Palestine fixed upon the Cassini geodetic projection with rectangular coordinates as calculated by Soldner as the projection for Palestine, based on the Jerusalem central meridian. The Cassini projection had been used by the British since 1745, and it was commended by the leading British survey This projection was considered easy for computation and suitable for areas of restricted size. From its geometrical attributes and its transverse construction, the Cassini projection answers the geodetic needs of Palestine within a strip 50–80 kilometres wide on both sides of a central meridian, usually passing through the centre of the area to be mapped , The British bestowed this honour on Jerusalem, so that the meridian became the central longitudinal line, even though it did not divide the country down the middle. The meridian of Jerusalem goes through the Jaffa Gate, and the main triangulation point82'M, which became the reference point of the system, was fixed higher up, on top of the Mar Elias monastery hill south of Jerusalem In the geodetic projection, importance is given not to the transfer of the elliptic geographic graticule of meridians and parallels, but to the replacement with a rectangular national grid system. The Surveys Directorate decided that the grid would encompass all the parts of the country to be mapped—which did not include the Negev south of Beersheba. Therefore, its staff established a trigonometrical station at the top of the 'Ali el-Muntar hill, which dominates the town of Gaza, in the heart of the area that was the first to be mapped in detail, and gave it values of 100–100 in the national grid. This point became the true origin of the Palestine grid [24].

In this way the zero point, or the false origin, of the Palestine axial system was 100 kilometers west and 100 kilometers south in north Sinai, near Jebel Maghara. The choice of the true point of origin was not a good one because it left the southern Negev with negative values south of the zero line. Thus, for example, Elat would have been given a negative northern coordinate of−116. In order to avoid negative values, the British set the value of the zero line at 1,000, so that anyplace south of the line would have positive values; Elat would thus be at 884 of the northern coordinate. When Richards conducted the check of the surveys in Palestine in 1925, he argued against this peculiar layout of the national grid. He remarked that the zero point of the main axes ought to have been at the intersection of the geographical coordinates 34° longitude and 29° latitude, which fall in south Sinai, so that all of Palestine would be within the positive values of the national grid , Richards also commented on the determination of the central meridian of the projection at Jerusalem, which it would have been better to move eastwards, for example to the Jordan Valley, so that in due course it would be possible to extend the grid system to Transjordan. These comments had no practical connotations, since the entire system was already in operation. The episode is mentioned here only to illustrate the absolute professional independence of the Directors of the Palestine Survey Department, despite the prestige of the Survey of gypt, which assisted the local department in its first steps [24].

The Parameter of the Palestine Projected Coordinate

Cassini_Soldner Projection The name Cassini-Soldner refers to the more accurate ellipsoidal version, developed in the 19 th century. This transverse cylindrical projection maintains scale along the central meridian and all lines parallel to it and is neither equal area nor conformal. It is most suited for large scale mapping of areas predominantly north-south in extent.785 which normally used in land surveying and engineering projects with the following parameters [25]:

Table (4-1) The Parameter Palestine (Grid-1923)

False Easting	170251.555000
False Northing	126867.909000
Central Meridian	35.212081
Scale Factor	1.000000
Latitude of Origin	31.734.097
Spheroid	Clarke_1880_Benoit
Semi major axis	6378300.790000000
Semi minor axis	6356566.430000036
Inverse flattening	293.46623457099997

Israel Old Grid is the same of Palestine grid (Paestine-1923-Grid), but 1 million is added to the northing value, because the coordinates of the south of Palestine) Al- Naqab (are negative, so it has been added 1 million to become All coordinates positive [25] .

Table (4-2) The Parameter Israel Old Grid

False Easting	170251.555000
False Northing	126867.909000
Central Meridian	35.212081
Scale Factor	1.000000
Latitude of Origin	31.734.097
Spheroid	Clarke_1880_Benoit
Semi major axis	6378300.790000000
Semi minor axis	6356566.430000036
Inverse flattening	293.46623457099997

4.5. GPS Control Network

GPS can be used like any other surveying tool; it can accomplish certain goals if we are conscious of its strengths and limitations. When surveying with GPS, we do not need to have inter-visibility between the stations to measure a baseline. The only constraint to receive the signals is having a clear view of the sky.

4.5.1 Principles for Establishment of GPS Control Networks

- **Establishment Based on Hierarchical Orders**

Setting GPS network into different orders is conductive to stage-wise establishment according to the immediate needs and long-term development of survey areas. Moreover, this principle enables the network structure to combine the long and short sides. Compared to the Short-Side GPS Control Network, the network established in such a way can reduce the accumulation of errors at its edge and allows data processing and results checking of GPS networks to be carried out easily in a piecemeal fashion.

For instance, we can first use GPS to establish a nationwide high- precision backbone control network with low density (A- and B-order networks or first and second-order networks) and then further densify the network using GPS or conventional methods based on the survey areas needed. In further densification, with the help of GPS technology it is unnecessary to establish an overall geodetic network in advance. Instead, one can establish and use the network at any time according to the accuracy required by users. We can obtain directly the known points from hundreds of kilometers away by GPS measurement, which not only saves a lot of manpower and material resources but also fulfills the practical needs [26].

- **Density**

Different task requirements and service targets have different requirements for establishing the GPS network. For example, the national super-network (AA-order) datum points are mainly used to provide national datums for orbit determination, precise ephemeris calculations, and large-scale ground deformation monitoring, with an average distance of hundreds of kilometers. The network required by general engineering survey with an average side length of several kilometers or even shorter (within hundreds of meters) should cater for the needs of mapping densification and engineering survey.

Taking the above factors into account, a rule for the distance between two adjacent points in GPS networks is made dependent on various needs: the average distance between adjacent points in GPS at all orders should meet the requirements of the data in table (4.1); the shortest distance between adjacent points can be 1/3 to 1/2 of the average distance while the longest is 2–3times. Under special circumstances, depending

on the network's task and target, the distance between some points can require specific rules for the distribution of GPS stations [26].

- **Accuracy**

In the design of GPS networks, the order and accuracy standard should be designed based on the size of survey areas and the use of the networks. The accuracy standard of general GPS measurement is commonly expressed by the mean square error of the distance between adjacent points in the networks as follows:

$$\sigma = \mp\sqrt{a^2 + (b.d)^2},$$

Where: σ is the mean square error of distance (mm), a is the constant error (mm), b is the coefficient of the ratio error, and d is the distance between adjacent points (km).The national “GPS survey specifications” classify GPS measurement into six orders, namely AA, A, B, C, D, E (as shown in table 4.1). The Table lists the distances between points and their accuracy indicators in GPS networks of different orders [23]

4.5.2 Technical Design of GPS Control Networks

- **Design of GPS Control Network Datum**

The design of the GPS control network datum is fundamental to the implementation of GPS measurement. It aims to find the best possible solution in terms of accuracy, reliability, and economic efficiency of the network. With GPS measurement we can obtain the GPS baseline vector between surface points, which belongs to the three- dimensional coordinate system of WGS84 or ITRF. Practical engineering applications require national coordinate systems like Beijing Coordinate System 1954, Xi'an Coordinate System 1980, China Geodetic Coordinate System 2000, or another independent local coordinate system. Therefore, in the technical design, the coordinate system and the initial data of the GPS network have to be specified, which means making clear the datum adopted by the GPS network [23].

The GPS network datum consists of position datum, azimuth datum, and scale datum. Position datum is usually determined by the coordinates of known initial points. Azimuth datum can be determined by the value of the known starting azimuth or the azimuth of the GPS baseline vector. Scale datum can be determined by the side of

the electromagnetic wave distance measurement on the Earth's surface, by the distance between two initial points, or by the distance of the GPS baseline vectors. So, the design of the GPS network datum is essentially the issue of determining the position datum of the network.

- **Point Selection**

Since GPS observation stations do not require indivisibility with each other, the selection of points is much simpler than for conventional measurements. The choice of GPS points has a significant influence on the smooth operation of GPS observations and the acquisition of reliable results. As a result, we should collect and fully understand the geographical conditions of the survey areas and the distribution and maintenance of existing control points based on the purpose of measurement and the requirements of coverage, accuracy, and density of the survey areas so as to properly choose the positions of the GPS points. The following principles should be followed in the selection of GPS point positions [23]:

1. It should be convenient to install antennae and GPS receivers around the point. The point should be located where the view is not obstructed and the elevation angle of the surrounding obstacles is less than 15^o.
2. The point should be far away from high-power radio emission sources and high voltage wires to avoid interference from magnetic fields close to the signal.
3. In order to weaken multipath effects, there should be no objects that strongly reflect or absorb electromagnetic waves around the point.
4. To improve operational efficiency, the point should be located where transportation is convenient.

Table (4.3.): Accuracy and density of GPS control networks

	Order					
Item	AA	A	B	C	D	E
Constant error *a* (mm)	≤3	≤5	≤8	≤10	≤10	≤10
Ratio error coefficient *b* (ppm)	≤0.01	≤0.1	≤1	≤5	≤10	≤20
Average distance between adjacent points (km)	1,000	300	70	15 ~ 10	10 ~ 5	5 ~ 0.2

5. Points should be selected taking into account the convenience of using other measurement techniques for connection and extension.
6. The point should be located in solid soil or, better, an outcrop of rock in order to be better preserved.
7. The integrity and stability of the survey mark should be checked before using old points.

Additionally, other conditions such as the nearby communication facilities and power supplies should also be considered for the connections between points and the electricity for equipment.

4.6 Marking the Position of the GPS Control Point

For long-term preservation, the GPS control point should usually be located on the survey mark (monument) with an identifier in the center to mark the point precisely.

Both the survey mark and the identifier should be stable and firm. The mark can be sunk into the ground or built into an observation stake or a stake with forced centering devices.

4.6.1 Monument for GPS Control Point

A large stone engraved with Circle and Dot, will be embedded in the ground firmly. It will act as a lower mark. A cement concrete pillar of dimension 1m×1m×1.2m will be constructed in such a manner that it is 0.7m below the ground level and 0.5m above ground. A „Survey Reference Mark made of Gun Metal/brass as shown in the figure (4.2) will be embedded at the center of pillar, flushed with the top surface of the pillar, see figure (4.2) [27]

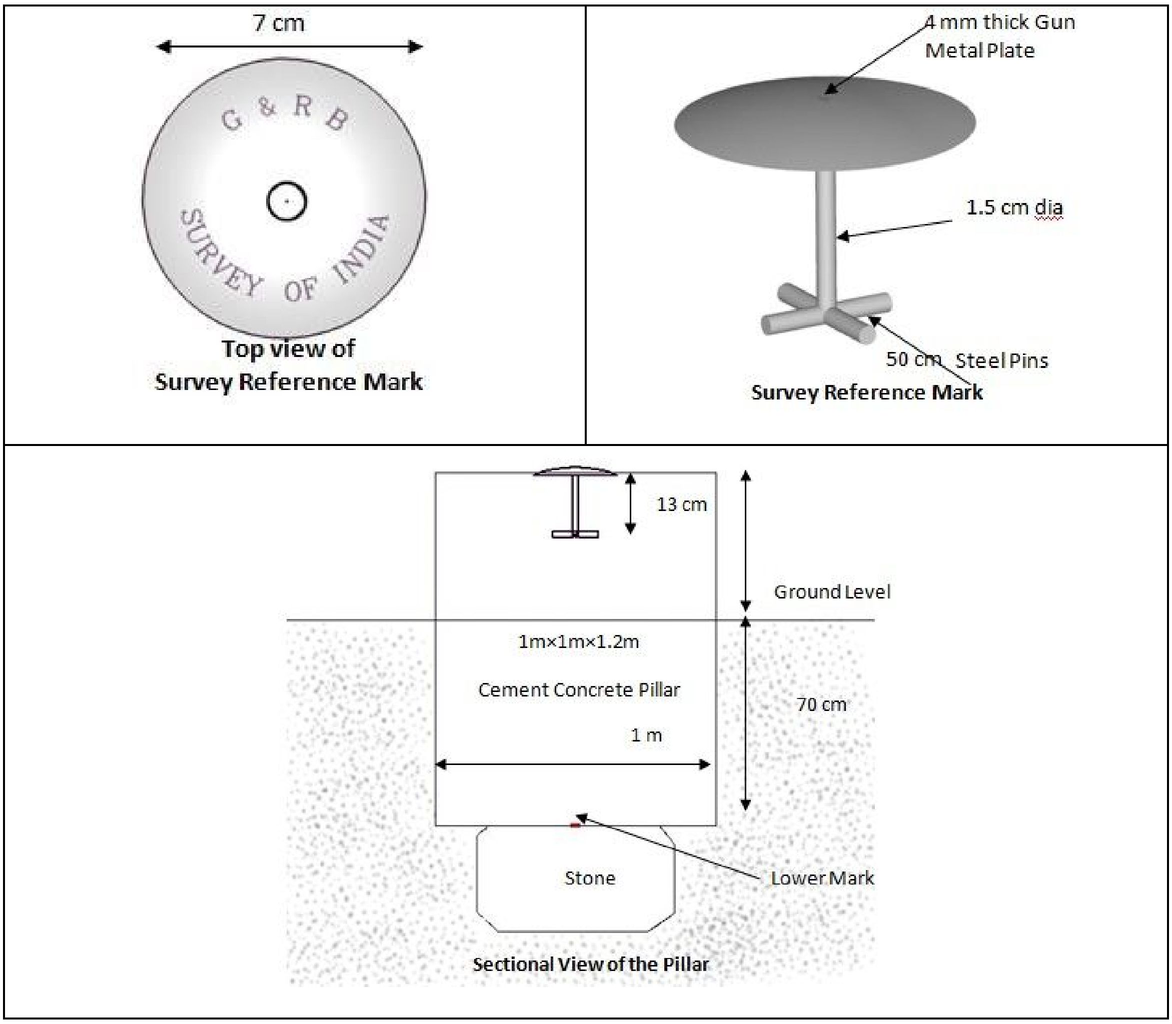

Figure (4.4): Design of a good GPS network observation scheme

4.6.2 Measurement Operations of GPS Control Networks

GPS measurement includes field observations and indoor data processing work. The former consists of installation of antennae, observation operations, and recording of field observation results whereas the latter consists of data extraction from instruments, baseline solution, and adjustment calculations of three- dimensional PS control networks, etc.

Below is an overview of field observations. For the extraction of observation data and the indoor work of data processing, please refer to annexed instructions and other relevant data [27].

- **Installation of Antennae**

The precise installation of antennae is one of the prerequisites for precise positioning and should satisfy the following conditions [27]:

1. Normally, the antenna should be installed in the vertical direction of the mark center on a tripod, directly centered. Only in exceptional cases can eccentric observations be allowed, where the centering elements should be precisely determined by analytical methods.
2. When installing an antenna on the platform of the tower, in order to avoid interference to signals, the top of the tower should be removed and the mark center be projected onto the platform. Then the antenna can be placed according to the projection point.
3. When there is an ordinary tower at the point and the distance between observation stations is less than 10 km, an antenna is allowed to be installed upon the tower, but the time of observation should be extended.
4. The pointer of the antenna should be directed to the true north. The effect of local magnetic declination should be considered and the orientation error should be no more than $\pm 5^{o}$.
5. The level bubble at the bottom of the antenna must be centered.
6. While installing an antenna in thunderstorm weather, the bottom of the antenna must be grounded to avoid lightning strikes. In a thunderstorm, observation operations should be stopped and the antenna removed.

After installation of the antenna, its height should be measured both before and after each time interval of observation. The difference between the results of the two measurements should be less than 3 mm. The average can be determined as the final height of the antenna. If the difference is larger than the tolerance, we should identify the causes, put forward suggestions, and note down the observation records.

The antenna height refers to the height from the average antenna phase center to the surface of the central mark of the observation station, which can be divided into two parts. One part covers the height from the phase center to the bottom of the antenna (i.e., the antenna reference point, ARP), which is a constant provided by manufacturers; the other part covers the height from the ARP to the surface of the central mark of the observation station, which should be measured by users on the spot. The specific measurement methods can be categorized into direct measurement and slant range measurement, according to the methods and types of antenna installation. Please refer to the receiver user manual for details. The final value of the antenna height is the sum of the heights of the parts [27].

- **Observational Operations**

Observational operations are mainly aimed at capturing, tracking, receiving, and processing GPS satellite signals to obtain the required data on positioning and observations. The operation of GPS receivers is highly automated. Its specific methods and procedures of operation vary with the types and operating modes of receivers.

Detailed information is included in the attached operation manuals. During operation, observers only need to follow the instructions in the operation manual. Generally, the following aspects should be noted [27]:

1. Observers at each receiver should work within the regulated observation time to ensure the realization of simultaneous observation of the same group of satellites.
2. After correct connection of the external power supply, the cable and antenna are confirmed, and the power can be switched on; when the receiver is in the right preset state it can be started.
3. When the data on the receiver's panel display is normal, observers can begin the self-test and input control information for observation stations and intervals of observation time.
4. When the receiver begins to record data, the observer should use function keys and selection menus to check information on observation stations, number of received satellites, satellite catalog number, channel SNR (signal-to-noise ratio), phase measurement residuals, results and changes in real-time positioning, records of storage media, etc.
5. During the period of observation, the receiver should not be turned off and restarted. The antenna height and limits of the elevation angle of satellites should not be changed. Observers are supposed to prevent vibration and especially displacement of the receiving devices. The antenna or signals should not be touched or obstructed.

6. When all the operation projects are confirmed to have been completed as required, the station can be moved.
7. In long-distance GPS measurements at higher levels, meteorological elements should be measured as required.

- **Observational Records**

Observational records are automatically formed by GPS receivers onto storage media, which include carrier phase observations, pseudo-range observations, corresponding GPS time, parameters of GPS satellite ephemeris, clock offset parameters, and initial information of observation stations such as name, catalogue number, time intervals, approximate coordinates, antenna height, and so on. The information on observation stations is generally first input by observers into the receivers or recorded manually in measurement handbooks [27].

4.7 Introduction to Network Adjustment

When performing network adjustments of GPS baselines, a least squares adjustment of the generated baselines is often performed once processing is complete and should follow the manufacturer's recommended procedures. These networks may comprise static and kinematic baselines. The network adjustment procedure has several functions in the GPS surveying process. The adjustment provides a single set of coordinates based on all the measurements acquired, as well as providing a mechanism by which baselines that have not been resolved to sufficient accuracy can be detected. A series of loop closures should be performed before the network adjustment procedure to eliminate erroneous baseline centering the adjustment process. A further feature of the network adjustment stage is the transformation parameters relating the GPS vectors to a local coordinate system can be estimated as part of the adjustment [26]. The adjustment

process can be done in several ways. The following sections highlight the major elements of the adjustment process.

4.7.1 Minimally Constrained Adjustment

Once the processed Cartesian vectors have been loaded into the adjustment module, an adjustment should be performed where one or no coordinates are constrained. The adjustment should be performed using the WGS84 datum and appropriate estimates of station centering error. This solution provides a mechanism by which GPS baselines, which are not sufficiently accurate, can be detected. Once the minimally constrained adjustment has been performed, the surveyor should analyze the baseline residuals and statistical outputs (which will differ between adjustment programs) and ascertain whether any baselines should be removed from subsequent adjustments. This process relies on the baseline network being observed in such a manner to ensure that redundant baselines exist. Redundant baselines enable erroneous baselines [26].

4.7.2 Fully Constrained Adjustment

Once the minimally constrained adjustment has been performed and all unsatisfactory baseline solutions removed, a fully constrained adjustment can be performed. The constrained adjustment is performed to compute transformation parameters, if required, and yield coordinates of all unknown points in the desired coordinate system. The surveyor must ensure that sufficient points with known coordinates are occupied as part of the survey. The user should analyze the statistical output of the processor to ascertain the quality of the adjustment. Large residuals at this stage, after the minimally constrained adjustment has been performed, will indicate that the control points are non-homogeneous. It is, therefore, important that additional control points are occupied to ensure that such errors can be detected [26].

4.7.3 Error Ellipses

The standard deviation of the estimated coordinates is derived from the inverse of the normal matrix generated during formulation of the least squares process. Error ellipses for each point can be computed from the elements of this matrix. The ellipse presents a two-standard deviation confidence region (95% certainty) in which the most probable solution based on the measurements will fall. Surveyors should base the quality of the adjustment process on the magnitude of these ellipses. Many contracts will specify the magnitude of error ellipses for both the minimally constrained and fully constrained adjustments as a method of prescribing required accuracy levels. The product documentation for the adjustment program will further indicate the manner in which the ellipse values are generated [26].

4.7.4 Independent Baselines (Non-Trivial Baselines)

For the least squares adjustment process to be successful, the surveyor must ensure that independent baselines have been observed. If more than one session is used to build the baseline network, then independent baselines will exist. In instances where one session is observed and all baselines adjusted, the measurement residuals will all be extremely small.

This is due to the correlation that exists between the baseline solutions as they are derived from common data sets. This is not a problem as long as the surveyor is aware of the occurrence and does not assume that the baselines are of as high accuracy as implied from the network adjustment results. For each observing session, there are n-1 independent baselines where n is the number of receivers collecting data simultaneously, with measurements inter-connecting all receivers during a session. If the mathematical correlation between two or more simultaneously observed vectors in a session is not carried in the variance-covariance matrix, the trivial baselines take on a bracing function simulating the effect of the proper correlation statistics, but at the same time introducing a false redundancy in the count of the degrees of freedom [26] .

4.7.5 Error Analysis

The local accuracies of property corners are based upon the results of a least squares adjustment of the survey observations used to establish their positions. They can be computed from elements of a covariance matrix of the adjusted parameters, where the known NSRS control coordinate values have been weighted using their one-sigma network accuracies.

The 95% confidence circle representing a local accuracy can be derived from the major and minor semi-axis of the standard relative ellipse between two selected points. It is closely approximated from the major (a) and minor (b) semi-axis parameters of the standard ellipse and a set of coefficients. For circular error ellipses, the circle coincides with the ellipse. Fore longated error ellipses, the radius of the circle will be slightly shorter than the major semi-axis of the ellipse. The radius of an error circle is equal to the major semi-axis of an associated error ellipse. The value of the largest error circle radius in the project should be adopted for reporting local project accuracy [5]

4.8 GPS Network Adjustments Procedures

4.8.1 Acquisition and check of GPS observation data

GPS observation data necessary for baseline processing and precise ephemeris of GPS satellite necessary for baseline processing of long baseline should preferentially be secured for GPS network adjustment. Besides, for accurate estimation of unknown point coordinate with GPS network adjustment, the process of checking the quality of observation data such as observed antenna height, observed station name, receipt interval of data & receipt time, receipt state of data is indispensable. Especially, in antenna height case, checkup of the location that measured APC(Antenna Phase Center), ARP(Antenna Reference Point), and of offset of antenna receiver by manufacturer is necessary. And a checkup of receiving time and state needs to be done by all means for the minute application of mathematical difference indifference to the observed value of carrier value obtained from the two points while base line processing. A checkup of receipt time & receipt state is required for the minute application of mathematical double difference to the carrier wave observed value gotten from the two points while baseline processing [5].

4.8.2 GPS baseline processing

GPS network is adjusted by using 3-dimensional baseline vector between two points generated from observed GPS data processing and VCV matrix. Baseline vector and VCV matrix can be calculated through GPS processing, a mathematical method and procedure estimating baseline vectors ΔX, ΔY, ΔZ between two points by least square method passing through the determination of integer ambiguity process after the application of mathematical double difference to the carrier wave observed value obtained from the two points at the same time effect of ionospheric and tropospheric, error of satellite track need to be fully checked up to raise the accuracy of baseline processing, and baseline processing S/W should be selected suitable for the purpose for use. Besides, after GPS processing of baseline vector needs to be examined if there exists outlier in baseline vector or not [5].

4.8.3 Minimally constrained adjustment

Minimally constrained adjustment is conducted to cover the following:

- To detect outlier in GPS network that hasn't been detected in the checkup that carried out after baseline processing,
- to calculate approximate accuracy of GPS network, and
- to determine the statistical model of GPS network that will be used in the final adjustment.

One point fixed adjustment and free adjustment are mainly used for minimally constrained adjustment, approximate accuracy of GPS network is assessed through RMSE or arithmetic mean through calculation of the size of relative error ellipse between points obtained after minimally constrained adjustment for certain probability (generally 95%) [5].

4.8.4 Outlier detection (tau-test)

In order to checkup outlier within GPS network, standardized residual should be calculated under the assumption that VCV matrix of measured value is unknown and then compare them with critical value determined by confidence interval and degree of freedom supposing they conform to students‟ distribution and then tau test was used for outlier detection. Taudistribution is used to estimate the confidence interval of the average of comparatively small mother group, and mainly used to check up the propriety of the average for certain part after comparison with the mother group average [18]

4.8.5 Empirical stochastic modeling

Random error components could be included in the corrected VCV matrix, but the errors included in the estimation of effect of atmosphere and antenna height estimation that haven't been fully modeled in baseline processing S/W couldn't be included in stochastic model, and so stochastic model is determined through Empirical stochastic modeling which corrects the variance, the diagonal element of VCV matrix by recalculating it for reflection of actual accuracy of GPS network. Empirical stochastic modeling was done in the way of changing and b in (4.1) by dividing it into horizontal(N, E) and vertical(H) until it passed χ2 test, a fidelity test for model. S is standard deviation of baseline vector, a is absolute error different to the points observed, b is relative error of ppm unit in proportion to baseline length, L is baseline length [5]

$$s^2_{E,N,H} = \left(a_{E,N,H} + b_{E,N,H} \cdot L\right) \qquad \textbf{(4-1)}$$

4.8.6 Propriety test for stochastic model ($\chi2$ test)

Stochastic test for variance factor is necessary for the propriety of mathematical model and stochastic model that used for estimation or detection of outlier, and for this test, attest for certain confidence needs to be carried out. The result can't pass the $\chi2$ test if inappropriate mathematical model or stochastic model was used or the measured value contains outlier.$\chi2$ test is the density function for the distribution of variance estimated from the degree of freedom selected by mother group, a critical value of distribution should be arranged into stochastic table for distribution use and stochastic test to determine confidence interval for the variance of mother group [18]

4.8.7 Over constrained adjustment

Over constrained adjustment is a way of determining the coordinate of unknown point and final adjustment results by using the coordinate of unknown point and the size of baseline vector between points & variation of VCV matrix. Function model, determined by relational expression of coordinate difference for each baseline made up of baseline vector is used for the determined coordinate of unknown point by over constrained adjustment. Function model could be formulated as observed equation and in case, the number of observed equation exceeds that of unknown quantity, solution could be obtained by least square method principle [5].

4.8.8 Checkup of adjusted result and assessment of accuracy

The comparison of check result with reliable accuracy and coordinate difference needs to be conducted along with comparison with individual applied point, and stochastic calculation such as RMSE, standard deviation, mean for the coordinate difference needs to be carried out for the checkup of adjusted results. Besides, the assessment of adjusted results accuracy is available through estimation of absolute error ellipsoid generated from GPS network adjustment. The accuracy of horizontal location of absolute error ellipsoid could be seen considering the correlation of east and west (E) and south and north (N), and the size and shape of error ellipsoid varies to the size of standard deviation of measured value and covariance between components [5].

Checkup and Analyze Loop Misclosure

Loop misclosure can be used to detect "bad" observations. (A bad observation can include misread antenna height, not being plumb over a point or observing the wrong point). If two loops with a common base line have large misclosure, this may be an indication that the common base line is an outlier. Since users must repeat base lines on different days and at different times of the day, there are several different loops that can be generated from the individual base lines. If a repeat base line difference is greater than 2 cm then comparing the loop misclosure involved with the base line may help determine which base line is the outlier. According to NGS guidelines, if a repeat base line difference exceeds 2 cm then one of the base lines must be observed again, and base lines must be observed at least twice on two different days and at two different times of the day [5].

4.9 Adjustment of GPS Network Models

For the past five decades, NASA and the US military participated in space research program a precise GPS navigation. System used six satellites of the first generation, called transit countries, and is based on the Doppler principle. Transit is available for commercial use in 1967, and shortly thereafter began to be used in the survey. The creation of a global network of control centers between applications at the earliest and the most valuable. GPS using transit required very long sessions control point, and accuracy to level 1-m. Thus, in the survey it was only suitable for business control over networks that consist of spaced points on a large scale. It was not satisfactory to clear applications such as engineering or planning transit every day.
Encouraged by the success of transit, a new research program developed which ultimately led to the creation of GPS in the NAVSTAR system (GPS). This system uses GPS and navigation of the second-generation constellation of 24 satellites orbits. Positioning accuracy have improved considerably compared to the transit system, depriving the long monitoring sessions, also canceled. Although the developed countries for military applications, and civilians, including surveyors, also found uses the global positioning system [9].

Since its introduction, it used widely GPS. They are reliable and efficient and able to generate a very high accuracy. Notes can be taken to determine the sites day or night and in any weather. A great feature for GPS it is not necessary to see the dots surveyed. Thus is avoided process takes a long time to scan the horizon lines.

Al- though most of the first applications of GPS in the work of monitoring, system improvements now comfortable and practical for use in almost every type of study, including surveys of ownership and topographic mapping, and staking of the building.

In practice, when surveys are done by observing carrier phases, four or more satellites are observed simultaneously using two or more receivers located on ground stations. Also, the observations are repeated many times.

This produces a very large number of redundant observations, from which many difference combinations can be computed .Of the two GPS observing procedures, pseudo ranging yields a somewhat lower order of accuracy, but it is preferred for navigation use because it gives instantaneous point positions of satisfactory accuracy. The carrier-phase technique produces a higher order of accuracy and is therefore the choice for high-precision surveying applications. The differencing techniques used in carrier-phase observations, described briefly above, do not yield positions directly for the points occupied by receivers. Rather, baselines (vector distances between stations) are determined. These baselines are actually computed in terms of their coordinate difference components ΔX, ΔY, and ΔZ. To use the GPS carrier-phase procedure in surveying, at least two receivers located on separate stations must be operated simultaneously. For example, assume that two stations A and B were occupied for an observing session, that station A is a control point, and that station B is a point of unknown position. The session would yield coordinate differences ΔX_{AB}, ΔY_{AB}, and ΔZ_{AB} between stations A and B. The X,Y,Z coordinates of station B can then be obtained by adding the baseline components to the coordinates of A [9] as

$$X_B=X_A+\Delta X_{AB}$$
$$Y_B= Y_A+ \Delta Y_{AB} \qquad (4.2)$$
$$Z_B= Z_A+ \Delta Z_{AB}$$

Because carrier-phase observations do not yield point positions directly, but rather, give baseline components, this method of GPS surveying is referred to as relative positioning. In practice, often more than two receivers are used simultaneously in relative positioning, which enables more than one baseline to be determined during each observing session. Also, after the first observing session, additional points are interconnected in the survey by moving the receivers to nearby stations. In this procedure, at least one receiver is left on one of the previously occupied stations. By employing this technique, a network of interconnected points can be created. Figure (4.3) illustrates an example of a GPS network. In this figure, stations A and B are

control stations, and stations C, D, E, and F: are points of unknown position. Creation of such networks is a common procedure employed in GPS relative positioning work.

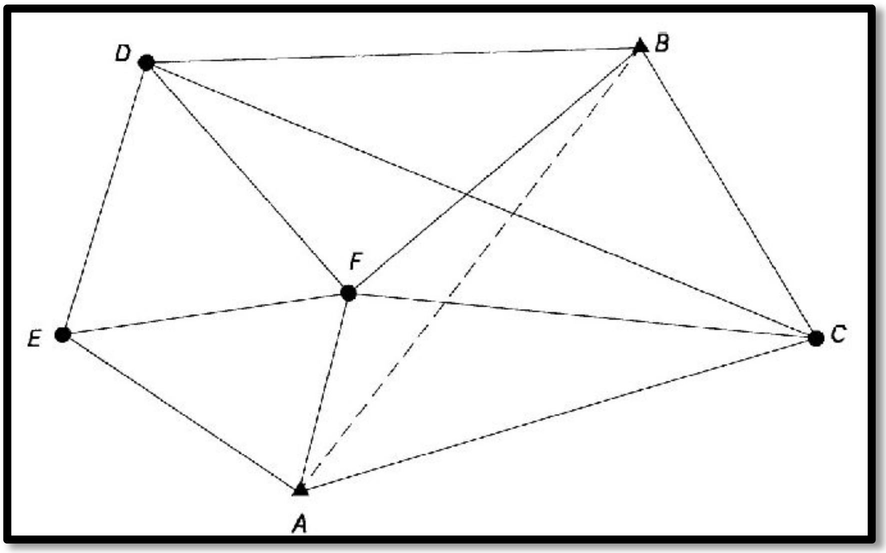

Figure (4.5): GPS survey network.

Chapter Five

Results and Analysis

The idea behind this chapter is to use a few fundamental concepts to help develop a way of thinking about GPS data processing. Yet has a firm theoretical foundation. Intuition is based on distilling an alarming array of information into a few core concepts that are basically simple. The fundamental concepts which that chosen to explore and develop here are generally based on equivalence principles and symmetry in problems. This involves looking at the same thing from different ways, or looking at apparently different things in the same way. Using symmetry and equivalence, we can often discover elegant explanations to problems. The ultimate goal is how to implementation the theoretical which was explained in previous chapters through a set of matrices and algorithm .

In the classical sense, a geodetic datum is a reference surface, generally an ellipsoid of revolution of adopted size and shape, with origin, orientation, and scale defined by a geocentric terrestrial frame. Once an ellipsoid is selected, coordinates of a point in space can be given in Cartesian or geodetic (curvilinear) coordinates (geodetic longitude, latitude, and ellipsoid height).

Two types of geodetic datum can be defined namely a static and kinematic geodetic datum. A static datum is thought of as a traditional geodetic datum where all sites are assumed to have coordinates which are fixed or unchanging with time. Most of traditional national geodesy grids are adopted on static datum such as Palestine grid 1923, this is an incorrect assumption since the surface of the earth is constantly changing because of tectonic motion. Static datum does not incorporate the effects of plate tectonics and deformation events, so the coordinates of Palestine geodesy is precisely defined at epoch of establishment, but slowly it will go out with date. need to change periodically which is disruptive.

In other hand the technique of GNSS is processing the inertial coordinates and transform those coordinate into terrestrial reference systems such as ITRS while the implementation of reference system is a reference terrestrial frame which is based on a kinematic geodetic datum.

Datum's can either become fully kinematic (dynamic), or semi-kinematic. A deformation model can be adopted to enable ITRF positions to be transformed into a static or semi-kinematic system at the moment of position acquisition so that users do not see coordinate changes due to global plate motions. GNSS devices which use ITRF or closely aligned systems position users in agreement with the underlying kinematic frame, however, in practice there are a number of very significant drawbacks to a kinematic datum. Surveys undertaken at different epochs cannot be combined or integrated unless a deformation model is applied rigorously, or is embedded within the data, and the data are correctly time-tagged. On the other hand, Semi–Kinematicdatum incorporates a deformation model to manage changes (plate tectonics and deformation events). Coordinates fixed at a reference epoch, so the change to coordinates is minimized. Many countries and regions which straddle major plate boundaries have adopted a semi-kinematic (or semi-dynamic) geodetic datum in order to prevent degradation of the datum as a function of time due to ongoing crustal deformation that is occurring within the country.

High precision GNSS positioning and navigation is very rapidly highlighting the disparity between global kinematic reference frames such as ITRF and WGS84, and traditional static geodetic datum. The disparity is brought about by the increasingly widespread use of PPP and the sensitivity of these techniques to deformation of the Earth due to plate tectonics. In order for precision GNSS techniques to continue to deliver temporally stable coordinates within a localized reference frame.

5.1 Transformation Parameters Terrestrial Reference Systems “TRS”

Recall that Transformations from kinematic ITRF to a static datum are conventionally done by either using the site velocity (measured directly or computed from a plate motion model) to compute the displacement between the reference and current epochs or by a conformal transformation augmented with time dependent parameters to account for rigid plate motion. Rigid Plate movement is conventionally defined by a rotation rate about an Euler Pole Φ Λ and , whereΦ, Λ are the latitude and longitude of the pole, and is the rate of rotation of the plate around the pole in degrees per million years. Equivalent rotation rates about the Cartesian axes (Ω_X Ω_Y andΩ_Z) can be computed from the Euler pole definition using equations below (5.1-5.3) (Φ,Λ, and $\boldsymbol{\omega}$) are first converted from decimal degrees to radians):

$$\Omega_X = \cos(\Phi)\cos(\Lambda)\,\omega \qquad (5.1)$$

$$\Omega_Y = \cos(\Phi)\sin(\Lambda)\,\omega \qquad (5.2)$$

$$\Omega_Z = \sin(\Phi)\,\omega \qquad (5.3)$$

A site velocity in Cartesian format ($X^{\cdot}$, $Y^{\cdot}$, $Z^{\cdot}$) can be computed for any given location (X, Y, Z in meters) on a rigid plate defined by (ΩX, ΩY, ΩZ in radians per million years) using:

$$\begin{bmatrix} X^{\cdot} \\ Y^{\cdot} \\ Z^{\cdot} \end{bmatrix} = \begin{bmatrix} \Omega yZ & - & \Omega zY \\ \Omega zX & - & \Omega xZ \\ \Omega xY & - & \Omega yX \end{bmatrix} .\ 1E^{-6} \qquad (5.4)$$

By introducing a reference epoch and an epoch of measurement t0 (epochs in decimal years), the ITRF coordinates of any point on a rigid plate at a reference epoch (X_0 ,Y_0 ,Z_0 in meters) can be computed from the coordinates at epoch t (Xt ,Yt ,Zt in meters) using:

$$\begin{bmatrix} X_O \\ Y_O \\ Z_O \end{bmatrix} = \left[\begin{bmatrix} X_t \\ Y_t \\ Z_t \end{bmatrix} + \begin{bmatrix} \Omega_{YZ_t} & - & \Omega_{ZYt} \\ \Omega_{zXt} & - & \Omega_{XZt} \\ \Omega_{XYt} & - & _{YXt} \end{bmatrix} (t_o - t) 1E^{-6} \right] \qquad (5.5)$$

For any location on a rigid plate, instantaneous ITRF coordinates can be transformed to a fixed reference epoch using equation (5-18) (Stanaway and Roberts, 2009).

$$\begin{bmatrix} X_O \\ Y_O \\ Z_O \end{bmatrix} = \begin{bmatrix} T_x \\ T_y \\ T_z \end{bmatrix} + S. \left[\begin{bmatrix} X_t \\ Y_t \\ Z_t \end{bmatrix} + \begin{bmatrix} \Omega_{YZ_t} & - & \Omega_{ZYt} \\ \Omega_{zXt} & - & \Omega_{XZt} \\ \Omega_{XYt} & - & _{YXt} \end{bmatrix} (t_o - t) 1E^{-6} \right] \qquad (5.6)$$

Where:

(X_O, Y_O, Z_O): are the ITRF Cartesian coordinates at the reference epoch t_o (in decimal years),

(Xt,Yt,Zt): are instantaneous ITRF Cartesian coordinates at epoch t (epoch in decimal years),

(Tx,Ty,Tz): are the translation of the reference frame origin (from ITRF to local system),

(Ωx, Ωy,Ωz): are the Cartesian rigid plate/block rotation parameters, is the reference frame

(S): is scale factor (from ITRF to local).

The transformation from ITRF to Palestine geodesy grid 1923 in this project was involved into two main procedures:

- Collecting GNSS data of locations for English triangulation network from second and third degree which had been distributed along areas of west bank.
- Applicate a calculations using software like GIS and EXCEL to set parameters of transformation and evaluate the Palestine geodesy grid 1923.

5.2 static mode of GNSS surveying and ppp

This method is used for surveying projects that require high accuracy. In this each receiver logs data at each point continuously for a pre-planned length of time and the duration of data collection depends upon required precision, number of visible satellites, satellite geometry (DOP), type of the receivers (single frequency or dual frequency) and distance between receivers.

Since 1994, the International GNSS Service (IGS) has provided precise GPS orbit products to the scientific community with increased precision and timeliness. Many national geodetic agencies and GNSS (Global Navigation Satellite System) users interested in geodetic positioning have adopted the IGS precise orbits to achieve centimeter level accuracy and ensure long-term reference frame stability

Relative positioning approaches that require the combination of observations from a minimum of two GNSS receivers, with at least one occupying a station with known coordinates are commonly used. The user position can then be estimated relative to one or multiple reference stations, using differenced carrier phase observations and a baseline or network estimation approach. Differencing observations is a popular way to eliminate common GNSS satellite and receiver clock errors. Baseline or network processing is effective in connecting the user position to the coordinates of the reference stations while the precise orbit virtually eliminates the errors introduced by the GNSS space segment. In our project we conducted survey of triangles Palestine grid 1923 by observe a interested points using one receiver (Leica 8 plus) and technique of PPP, where this receiver was on contact with IGS stations called RAMO, ANQARA and nearby IGS stations.

One drawback is the practical constraint imposed by the requirement that simultaneous observations be made at reference stations. An alternative post-processing approach uses un-differenced dual-frequency pseudo-range and carrier phase observations along with IGS precise orbit products, for stand-alone precise geodetic point positioning (static or kinematic) with centimeter precision. This is possible if one takes advantage of the satellite clock estimates available with the satellite coordinates in the IGS precise orbit/clock products and models systematic effects that cause centimeter variations in the satellite to user range. Furthermore, station tropospheric zenith path delays with mm precision and GNSS receiver clock estimates precise to 0.03 nanosecond are also obtained. To achieve the highest accuracy and consistency, users must also implement the GNSS-specific conventions and models adopted by the IGS. This paper describes both post-processing approaches, summarizes the adjustment procedure and specifies the Earth and space based models and conventions that must be implemented to achieve mm-cm level positioning, tropospheric zenith path delay and clock solutions.

Acquiring of Static data and files from receiver

The filed work include inspect of ensuring existing a locations a triangles of Palestine grid 1923 which have classify from second and third degree, and observe them using (method of a static observation) at least two hours by technique of PPP. All interesting points in our project of Palestine grid 1923 were visited and collecting data about them like location photo of triangles, some information …….etc. , that was prepared in appendix with book of project .

Each existing point was monitored by receiver from type (Leica 8 plus) and they have separated folder in data collector, those folder have a extend of .DBX and consist from 8 files .the file which have an extension of .m00 it's able to convert to RINEX by using software which called TEQC

the previous processing can be available direct from data collectors to getting on RINEX files, but isn't available on the pocket-pc because most of surveying works here in Palestine limited in method of RTK, so the supplier of GPS instruments may didn't install a specialize software on pocet_pc.

Converting RINEX

Receiver INdependent EXchange format files contain raw satellite navigation system data relative to a specified interval of time (typically one calendar day). They allow users to add corrections to their data in post-processing, improving its accuracy. The RINEX version 3.00 format consists of three ASCII file types Observation data File, Navigation message File, Meteorological data File.

The RINEX have extension filenames: ssssdddhmm.yyO where:

- O: observation file
- yy: two-digit year
- mm: starting minute within the hour (00, 15, 30, 45)
- h: character for the n-th hour in the day
- a = 1st hour: 00h-01h; b = 2nd hour: 01h-02h
- . . . x = 24th hour: 23h-24h.
- ddd: day of the year
- ssss: 4-char station ID or ID for the LEO receiver/antenna

We can getting on a geocentric Cartesian coordinates from RINEX after uploading those files on a special website to processing them to be available for studies of geodesy, there is many website which are deal with RINEX formats such as TRIMBLE, so this websites converting RINEX data file to geocentric coordinates according to chosen dynamic geodetic datum such as ITRF and WGS84 and sending them in reports include the errors and standard deviation of converting.

In our project were uploading data and processing them using three website based on choosing a geodetic datum different per site where:

http://www.ga.gov.au/scientific-topics/positioning-navigation/geodesy/auspos
https://www.ngs.noaa.gov/OPUS/
http://trimblertx.com/

Summary of observations

Geocentric Cartesian									STANDARD DEVIATION		
Trig Name		X (m)	Y (m)	Z (m)	START TIME	END TIME	DATE	DURATION	σ x	σ y	σ z
1	148 T	4391296.089	3116624.126	3407256.946	17:02:46 UTC	19:05:46UTC	7/7/2017	2 H	0.050	0.071	0.027
2	148 T /1H	4391296.071	3116624.147	3407256.936	15:58:54 UTC	16:51:02UTC	7/7/2017	1 H	0.025	0.031	0.013
3	91 M	4414664.092	3126952.903	3368808.412	4:35:56 UTC	6:35:59 UTC	7/7/2017	2 H	0.047	0.065	0.017
4	520 T	4449770.582	3124401.353	3324883.885	16:03:56 UTC	6:04:00 UTC	8/10/2017	2 H	0.023	0.024	0.018
5	541 T	4452006.449	3134300.147	3312749.459	08:57:42 UTC	11:06:14UTC	8/9/2017	2 H	0.020	0.020	0.012
6	531 B	4456152.498	3119662.722	3319155.520	10:30:20 UTC	12:40:14UTC	9/10/2017	2 H	0.032	0.022	0.019
7	606 M	4411315.099	3107878.467	3390030.295	10:25:40 UTC	12:30:39UTC	7/7/2017	2 H	0.024	0.030	0.014
8	617 M	4412717.237	3143202.721	3354317.760	22:15:50 UTC	00:30:36UTC	7/7/2017	2 H	0.022	0.018	0.011

Table (5.1) Geocentric Cart esian coordinates ITRF 2014 @ EPOCH 2017.51from TRIMBLE reports

Geocentric Cartesian									STANDARD DEVIATION		
Trig Name	X (m)	Y (m)	Z (m)	START TIME	END TIME	DATE	DURATION	σ x	σ y	σ z
1	148 T	4391296.259	3116624.034	3407256.811	17:02:46 UTC	19:05:46UTC	07/07/2017	2 H	0.050	0.071	0.027
2	148 T /1H	4391296.241	3116624.055	3407256.801	15:58:54 UTC	16:51:02UTC	07/07/2017	1 H	0.025	0.031	0.013
3	91 M	4414664.262	3126952.810	3368808.276	4:35:56 UTC	6:35:59 UTC	07/07/2017	2 H	0.047	0.065	0.017
4	520 T	4449770.752	3124401.257	3324883.747	16:03:56 UTC	6:04:00 UTC	10/08/2017	2 H	0.023	0.024	0.018
5	541 T	4452006.619	3134300.052	3312749.321	08:57:42 UTC	11:06:14UTC	09/08/2017	2 H	0.020	0.020	0.012
6	606 M	4411315.268	3107878.374	3390030.160	10:25:40 UTC	12:30:39UTC	07/07/2017	2 H	0.024	0.030	0.014
7	617 M	4412717.435	3143202.638	3354317.579	22:15:50 UTC	00:30:36UTC	07/07/2017	2 H	0.022	0.018	0.011

Table (5.2) Geocentric Cartesian coordinates ITRF 2014 @ EPOCH 2010 from TRIMBLE reports

Geocentric Cartesian									STANDARD DEVIATION		
Trig Name		X (m)	Y (m)	Z (m)	START TIME	END TIME	DATE	DURATION	σ x	σ y	σ z
1	148 T	4391296.569	3116623.888	3407256.565	17:02:46 UTC	19:05:46UTC	7/7/2017	2 H	0.025	0.031	0.013
2	148 T /1H	4391296.550	3116623.909	3407256.556	15:58:54 UTC	16:51:02UTC	7/7/2017	1 H	0.050	0.071	0.027
3	91 M	4414664.570	3126952.662	3368808.030	4:35:56 UTC	6:35:59 UTC	7/7/2017	2 H	0.047	0.065	0.017
4	520 T	4449771.059	3124401.107	3324883.499	16:03:56 UTC	6:04:00 UTC	8/10/2017	2 H	0.023	0.024	0.018
5	541 T	4452006.926	3134299.901	3312749.073	08:57:42 UTC	11:06:14UTC	8/9/2017	2 H	0.020	0.020	0.012
6	606 M	4411315.577	3107878.226	3390029.914	10:25:40 UTC	12:30:39UTC	7/7/2017	2 H	0.024	0.030	0.014
7	617 M	4412717.792	3143202.506	3354317.253	22:15:50 UTC	00:30:36UTC	7/7/2017	2 H	0.022	0.018	0.011

Table (5.3) Geocentric Cartesian coordinates ITRF 1997 @ EPOCH 1997 from TRIMBLE reports

5.3 Helmert transformation 7 parameters

The Helmert 7 parameter transformation offers a mathematically correct transformation. This maintains the accuracy of the GPS measurements and local coordinates. Experience has shown that it is common for GPS surveys to be measured to a much higher accuracy than older surveys measured with traditional optical instruments In the vast majority of cases, the previously measured points will not be as accurate as the new points measured with GPS. This may create non-homogeneity in the network.

When transforming a point between coordinate systems, it is best to think of the origin from which the coordinates are derived as changing and not the surface on which it lies. In order to transform a coordinate from one system to another, the origins and axes of the ellipsoid must be known relative to each other. From this information, the shift in space in X, Y and Z from one origin to the other can be determined, followed by any rotation about the X, Y and Z axes and any change in scale between the two ellipsoids.

The Helmert transformation is defined by Equation 1, relating the geocentric Cartesian co-ordinates
(XA, YA, ZA) in one system to co-ordinates (XB, YB, ZB) in another system. The three components of
The Helmert transformation (scale, rotations and shifts). Note that this the assumption

That the rotations are small; if this is not the case, a full orthogonal rotation matrix is required.

$$\begin{bmatrix} X_B \\ Y_B \\ Z_B \end{bmatrix} = \begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix} + \begin{bmatrix} \Delta_X \\ \Delta_Y \\ \Delta_Z \end{bmatrix} + \begin{bmatrix} \mu & \theta_Z & -\theta_Z \\ -\theta_Z & \mu & \theta_X \\ \theta_Y & -\theta_X & \mu \end{bmatrix} * \begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix} \tag{5.7}$$

- XA, YA, ZA = geocentric Cartesian co-ordinates in system A
- XB, YB, ZB = geocentric Cartesian co-ordinates in system B
- θX, θY, θZ = small rotations (in radians) about axes of system A
- Δ_X, ΔY, ΔZ = linear shifts along axes of system A
- μ = scale change between two systems

The reverse Helmert formula is:

$$\begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix} = \begin{bmatrix} X_B \\ Y_B \\ Z_B \end{bmatrix} - \begin{bmatrix} \Delta_X \\ \Delta_Y \\ \Delta_Z \end{bmatrix} - \begin{bmatrix} \mu & \theta_Z & -\theta_Z \\ -\theta_Z & \mu & \theta_X \\ \theta_Y & -\theta_X & \mu \end{bmatrix} * \begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix}$$

(5.8)

To finding and evaluate the seven unknown parameters from the equation 5.7, at least we need three control points known in both systems, and when the number of known control points are more than three then the least square solution will be more accurate.

Methodology

The method for computing the Helmert transformation parameters relating the legacy and new coordinate systems was as outlined below. Each stage is described in more detail in the following Sections.

Obtain two sets of Cartesian coordinates for the network stations, one based on the legacy system and one on the new, GPS-based system
Compute, test and update Helmert transformation parameters.

Before calculations procedure we must process coordinates data of legacy system which it represent a Palestine grid 1923, where the available data about those points is projection coordinates (EASTING AND NORTHING), so we must convert it to geographic geocentric coordinates (Φ λ and h) respect to the Clarck 1880 parameters which it a reference ellipsoid

of Palestine grid 1923. Then it must to convert it to geocentric Cartesian coordinates (X Y and Z) to calculate a seven parameters using Helmert formula. In our process of converting from projection coordinates to geographic coordinates were adopted on ARCGIS software.

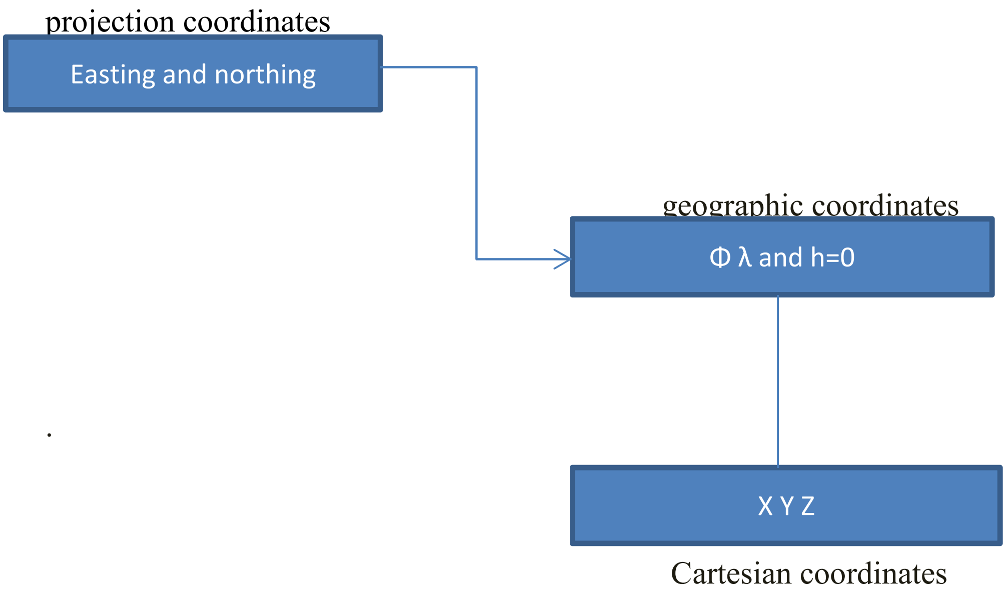

Figure (5.3) conversion of legacy coordinates

Cassini projection formula

The Cassini-Soldner projection is the ellipsoidal version of the Cassini projection for the sphere. It is not conformal but as it is relatively simple to construct it was extensively used in the last century and is still useful for mapping areas with limited longitudinal extent. It has now largely been replaced by the conformal Transverse Mercator which it resembles. Like this, it has a straight central meridian along which the scale is true, all other meridians and parallels are curved, and the scale distortion increases rapidly with increasing distance from the central meridian where :

EASTING = FE + $\boldsymbol{\nu}\left[\boldsymbol{A} - \frac{\boldsymbol{TA^3}}{\boldsymbol{6}} - (\boldsymbol{8} - \boldsymbol{T} + \boldsymbol{8C})\boldsymbol{TA^5}/\boldsymbol{120}\right]$
(5.9)

NORTHING = FN + M - Mo + $\boldsymbol{\nu tan}[\boldsymbol{A}\text{^}\boldsymbol{2}/\boldsymbol{2} + (\boldsymbol{5} - \boldsymbol{T} + \boldsymbol{6C})\boldsymbol{A^4}/\boldsymbol{24}]$ (5.10)

where $A = (\lambda - \lambda O)\cos$

$T = \tan^2 \Phi$

$C = e^2 \cos^2 \Phi / (1 - e^2)$

$\nu = a / (1 - e^2 \sin 2 \Phi)\ 0.5$

and M, the distance along the meridian from equator to latitude ϕ, is given by $M = a[1 - e^2/4 - 3e^4/64 - 5e^6/256 - \ldots) - (3e^2/8 + 3e^4/32 + 45e^6/1024 + \ldots)\sin^2 \Phi + (15e^4/256 + 45e^6/1024 + \ldots)\sin 4\Phi - (35e^6/3072 + \ldots)\sin 6\Phi + \ldots]$

with Φ in radians.

To compute latitude and longitude from Easting and Northing the reverse formulas are:

$\Phi = \Phi 1 - (\nu 1 \tan\ 1/\rho 1)[D^2/2 - (1 + 3T1)D^4/24]$ (5.11)

$\lambda = \lambda O + [D - T1D^3/3 + (1 + 3T1)T1D^5/15]/\cos\ 1$ (5.12)

Where

$\nu 1 = a / (1 - e^2 \sin^2 \Phi 1)^{0.5}$ (5.13)

$\rho 1 = a(1 - e^2)/(1 - e^2 \sin^2 \Phi 1)^{1.5}$ (5.14)

Φ 1 is the latitude of the point on the central meridian which has the same Northing as the point whose coordinates is sought, and is found from:

$\Phi 1 = \mu 1 + (3e1/2 - 27e1^3/32 + ...)\sin^2\mu 1 + (21e1^2/16 - 55e1^4/32 + ...)\sin 4\mu 1 + (151e1^3/96 + ...)\sin 6\mu 1 + (1097e1^4/512 - ....)\sin 8\mu 1 + ...$

Where
$e1 = [1 - (1 - e^2)^{0.5}]/[1 + (1 - e^2)^{0.5}]$ $\mu 1 = M1/[a(1 - e^2/4 - 3e^4/64 - 5e^6/256 - ....)]$ $M1 = MO + (N - FN)$ $T1 = \tan^2 \Phi 1$ $D = (E - FE)/\nu 1$

Semi-major axis, a	6378300.782	True origin latitude, φ_0	N	31	44	2.7
Semi-minor axis, b	6356566.42744003	True origin longitude, λ_0	E	35	12	43
Central Meridan Scale, F_0	1.000000000000					
True origin Easting, E_0	170251.555					
True origin Northing, N_0	126867.909					

Table (5.4) Clarck 1880 ellipsoid parameters

point	X	Y	Φ	λ	Φ in radians	λ in radians
148 T	184487.06	211506.32	32.497584	35.364382	0.5671898397	0.6172249038
91 M	179450.1	165624.82	32.083898	35.310306	0.5599696570	0.6162810996
520 T	157133.47	113959.94	31.617948	35.074578	0.5518372953	0.6121668698
541 T	163931.55	99679.27	31.489208	35.146304	0.5495903584	0.6134187247
531 B	149580.58	107780.39	31.562111	34.995102	0.5508627558	0.6107797520
606 M	165800.751	190898.815	32.311851	35.165631	0.5639481874	0.6137560445
617 M	193856.483	149216.759	31.935712	35.462487	0.5573833234	0.6189371591

Table (5.4) conversion of projection coordinate to geographic coordinates

Formulas for conversion geographic coordinates to Cartesian:

$$X = (\nu + h)\cos\phi\cos\lambda \quad (5.15)$$

$$Y = (\nu + h)\cos\phi\sin\lambda \quad (5.16)$$

$$Z = ((1 - e^2)\nu + h)\sin\phi \quad (5.17)$$

Where ν is the prime vertical radius of curvature at latitude ϕ and is equal to
ν = a /(1 – e2 sin2 ϕ) 0.5,

ϕ and λ are respectively the latitude and longitude (related to the prime meridian) of the point,

h is height above the ellipsoid, (see note below), and
e is the eccentricity of the ellipsoid where e2 = (a2 – b2)/a2 = 2f – f2

point		Φ	λ	dd	mm	ss.ssss	dd	mm	ss.ssss	ellipsoidal hieght	X	Y	Z
1	148 T	32.497584	35.364382	32	29	51.302400000015	35	21	51.775199999640	0.000	4391088.1250	3116477.3631	3407097.2011
2	91 M	32.083898	35.310306	32	5	2.032799999637	35	18	37.101599999631	0.000	4414025.8801	3126496.6821	3368317.4194
3	520 T	31.617948	35.074578	31	37	4.612799999639	35	4	28.480800000005	0.000	4449165.3380	3123977.5192	3324430.7792
4	541 T	31.489208	35.146304	31	29	21.148800000010	35	8	46.694400000008	0.000	4451359.0416	3133844.9006	3312266.6933
5	531 B	31.562111	34.995102	31	33	43.599600000001	34	59	42.367200000008	0.000	4456151.1956	3119662.9819	3319157.0137
6	606 M	32.311851	35.165631	32	18	42.663599999636	35	9	56.271599999629	0.000	4410895.4961	3107583.6342	3389707.7825
7	617 M	31.935712	35.462487	31	56	8.563199999640	35	27	44.953200000004	0.000	4412800.3838	3143262.8123	3354383.8736

Table (5.5) conversion geographic coordinate to geocentric Cartesian coordinates

Since we are converting the projection coordinates to Cartesian coordinates also the coordinate data of a new system must be processed to be in the similar plane with the legacy system because the legacy system have 2D coordinates, so the Cartesian coordinates must be converting to geographic coordinates and assume the ellipsoid high = 0 and exchange it again to Cartesian coordinates.

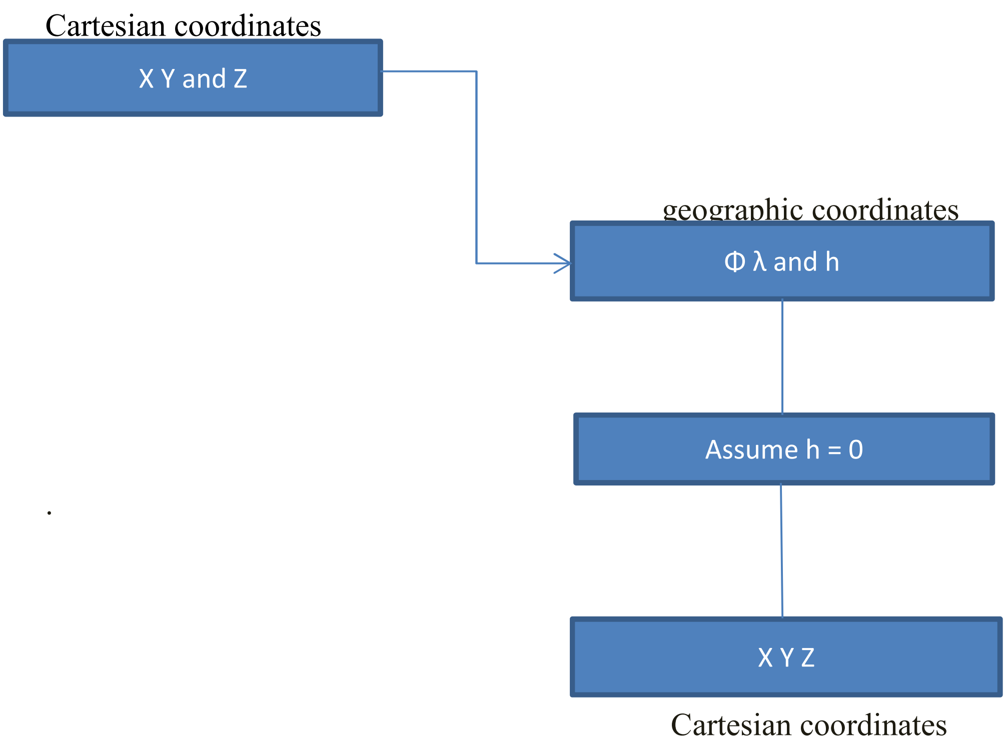

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Figure (5.5) conversions a new coordinate system

Triq Name		Geographical							Geocentric Cartesian		
		Latitude (N)			Longitude (E)			Ellipsoidal	X	Y	Z
		dd	mm	ss.ssss	dd	mm	ss.ssss	height (m)	(m)	(m)	(m)
1	148 T	32	29	51.236786713995	35	21	51.749120404656	0.000	4391089.4046	3116477.4364	3407095.4964
2	148 T /1H	32	29	51.236556958120	35	21	51.750175432644	0.000	4391089.3917	3116477.4610	3407095.4905
3	91 M	32	5	2.010352305933	35	18	37.231319497137	0.000	4414024.2135	3126499.6703	3368316.8335
4	520 T	31	37	4.563596178804	35	4	28.445475606282	0.000	4449166.5232	3123977.2138	3324429.4887
5	541 T	31	29	21.092674836468	35	8	46.677634671037	0.000	4451360.0347	3133845.0586	3312265.2192
6	531 B	31	33	43.542700000000	34	59	42.330800000000	0.000	4456152.4976	3119662.7216	3319155.5205
6	606 M	32	18	42.613331058238	35	9	56.246987451295	0.000	4410896.5436	3107583.5846	3389706.4738
7	617 M	31	56	8.503389087792	35	27	44.926364844601	0.000	4412801.5865	3143262.8036	3354382.3102

Table (5.6) conversions Geographical coordinates to Geocentric Cartesian

Applying Helmert transformations

Calculations excluded M points

v=
0.01209703
0.037812746
-0.050887312
0.01778638
-0.194433939
0.158643312
-0.183264677
0.304437518
-0.045437514
0.153381265
-0.147816325
-0.062318485

Legacy Coordinates AFTER APPLYING PARAMETERS			
point	X0	Y0	Z0
148 T	4391088.137	3116477.401	3407097.15
520 T	4449165.356	3123977.325	3324430.938
541 T	4451358.858	3133845.205	3312266.648
531 B	4456151.349	3119662.834	3319156.951
606 M	4410895.325	3107583.563	3389708.073
617 M	4412800.325	3143262.897	3354383.857

parameters after six iterations	
S	1.000002
W	4.89E-05
ɸ	1.89E-05
K	5.77E-05
Tx	-9.44222
Ty	-8.78926
Tz	-7.46813

applying parameters on a excluded points from calculations of parametrs			
POINT	X	Y	Z
606 M	4410896.544	3107583.585	3389706.474
617 M	4412801.586	3143262.804	3354382.310

Calculations included M POINTS

v=
0.12204731
-0.026379161
-0.132216588
0.004064127
-0.036876596
0.029199342
-0.076483225
-0.078568136
0.169276929
-0.049628208
0.141823893
-0.066259682

Legacy Coordinates AFTER APPLYING PARAMETERS			
point	X0	Y0	Z0
148 T	4391088.247	3116477.337	3407097.069
520 T	4449165.342	3123977.482	3324430.808
606 M	4410895.42	3107583.556	3389707.952
617 M	4412800.334	3143262.954	3354383.807
541 T	4451358.818	3133845.387	3312266.536
531 B	4456151.332	3119663.01	3319156.805

parameters after six iterations	
S	1.000002
W	0.000159
ϕ	9.28E-05
K	0.000121
Tx	-9.63358
Ty	-6.67906
Tz	-7.35955

applying parameters on a excluded points from calculations of parametrs			
POINT	X	Y	Z
541 T	4451360.035	3133845.059	3312265.219
531 B	4456152.498	3119662.722	3319155.520

5.4 Miscellaneous Linear Coordinate Operations (2D AFFINE)

An affine 2D transformation is used for converting or transforming a coordinate reference system
Possibly with non-orthogonal axes and possibly different units along the two axes to an isometric
Coordinate reference system (i.e. a system of which the axes are orthogonal and have equal scale units). The transformation therefore involves a change of origin, differential change of axis orientation and a differential scale change. Widespread example for explaining this theory a tool in ARCMAP for georeferencing and align command in Autodesk software.

The formula for affine transformation is described in matrix as follow:

$$A\ matrix = \begin{matrix} e1 & n1 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & e1 & n1 & 1 \\ e2 & n2 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & e2 & n2 & 1 \\ e3 & n3 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & e3 & n3 & 1 \end{matrix} \qquad (5.18)$$

Where e,n are legacy coordinates (Palestine grid 1923)

$$Lmatrix = \begin{matrix} E1 \\ N1 \\ E2 \\ N2 \\ E3 \\ N3 \end{matrix} \qquad (5.19)$$

Where E,N are projected coordinates from kinematic ITRF to clarck 1880 ellipsoid to cassini .

$$\text{X matrix} = \begin{matrix} a \\ b \\ c \\ d \\ e \\ f \end{matrix} \qquad (5.20)$$

The relation to determine unknown transformation parameters a,b,c,d,e and f is :

$$X = (A^T A)^{-1} * A^T L \qquad (5.21)$$

An error matrix is defines as:

$$V = AX - L \quad (5.22)$$

To convert from new coordinate system to legacy as follow:

$$E_{LEGACY} = aE_{new} + bN_{new} + c \quad (5.23)$$
$$N_{LEGACY} = dE_{new} + eN_{new} + f \quad (5.24)$$

The coordinates in new systems must be converted from geographic coordinates to projected 2D coordinates that was done using software of ARCGIS.

TRIG NAME	gegraphic coordinates		projection to cassini from ITRF 2014@EPOCH 2017.51		legacy coordinates at palestine gird 1923	
	Φ	λ	EASTING	NORTHING	EASTING	NORTHING
148 T	32.497584	35.364382	184486.398	211504.315	184487.060	211506.320
91 M	32.083898	35.310306	179453.504	165624.138	179450.100	165624.820
520 T	31.617948	35.074578	157132.509	113958.465	157133.470	113959.940
541 T	31.489208	35.146304	163931.060	99677.584	163931.550	99679.270
606 M	31.562111	34.995102	165800.114	190897.256	165800.751	190898.815
617 M	32.311851	35.165631	193855.740	149214.943	193856.483	149216.759
531 B	31.935712	35.462487	149579.595	107778.642	149580.580	107780.390

Table (5.7). Projected coordinates of new system

Results of (2D AFFINE)

X MATRIX PARAMETERS VALUE	
a	1.000001016
b	2.48126E-07
c	-0.912361711
d	-9.95461E-06
e	0.999999191
f	0.138138779

V MATRIX
-0.011
0.136
0.236
-0.044
-0.231
0.112
-0.059
-0.107
0.065
-0.097

Chapter six

Conclusions and Recommendations

6.1 Results

A geodetic control network is the wire-frame or the skeleton on which continuous and consistent mapping, Geographic Information Systems (GIS), and surveys are based. Traditionally, geodetic control points are established as permanent physical monuments placed in the ground and precisely marked, located, and documented. With the development of satellite surveying methods and their availability and high degree of accuracy, a geodetic control network could be established by using GNSS and referred to an international terrestrial reference frame used as a three-dimensional geocentric reference system for a country. Based on this concept This research work was conducted as part of the project.

- Was observed 8 triangulation point according to which coordinate values are calculated in the (Palestine Grid 1923) based on the Cassini-Soldner. Was established during the British Mandate period covering all the Palestinian territories.

- Great number of the triangulation point Can be classified as 'Destroyed', 'Possibly Missing' or 'Inaccessible' for whatever reason.

- Second-grade missing points were subrogated by the third grade from the Palestine Grid 1923.

- Precise Point Positioning (PPP) satellite based positioning technique has been adopted because it provides a positioning solution in a dynamic, global reference frame such as the International Terrestrial Reference Frame (ITRF).

- Four PPP post-processing services was used a matched result were achieved however, Trimble service results were adopted for the multiple options provided ITRF year and the Epoch details .

- Because the geographic coordinate systems contain datums that are based on spheroids, Transformation procedures taken as the following:

(A)

1- ITRF $\xrightarrow{\text{Clark}}$ λ , φ , h $\xrightarrow{\text{Clark}}$ X , Y , Z

2- E , N $\xrightarrow{\text{Clark}}$ λ , φ , h $\xrightarrow{\text{Clark}}$ X , Y , Z

3- conformal transformation adopted (Helmert transformation) accepting that the accuracy will be degraded since datum variations are modelled by an 'average' transformation.

(B)

1- ITRF $\xrightarrow{\text{Clark}}$ λ , φ , h $\xrightarrow{\text{Cassini}}$ E , N

2- 2D Affine transformation to Palestine Grid 1923.

	Max of residual	Min of residual	Slandered error
Helmert	0.1692	0.0040	0.6083
2D-Affine	0.2360	-0.011	0.1771

Table (6-1) Comparison Of Transformation Methods Used

- The orders of the Networks are defined By the Federal Geodetic control Subcommittee for both traditional and GPS surveys as [28]:

GPS Order*	Traditional Surveys Order and Class**	Relative Accuracy Required Between Points
Order AA		1 part in 100,000,000
Order A		1 part in 10,000,000
Order B		1 part in 1,000,000
Order C-1	First Order	1 part in 100,000
	Second Order	
Order C-2-I	Class I	1 part in 50,000
Order C-2-II	Class II	1 part in 20,000
	Third Order	
Order C-3	Class I	1 part in 10,000
	Class II	1 part in 5000

Table (6.2) Distance accuracy standards

148 T - 520 T	101308.5779	101309.0082	-0.430249503	-4.24692E-06	-235464.718
148 T - 541 T	113700.2182	113700.5633	-0.345164889	-3.03575E-06	-329408.4123
148 T - 606 M	27817.76628	27818.11292	-0.346635658	-1.24609E-05	-80250.73491
148 T - 617 M	62990.08154	62990.28097	-0.199434525	-3.16613E-06	-315843.4161
148 T - 531 B	109441.7652	109441.9065	-0.14129606	-1.29106E-06	-774556.382
520 T -541 T	15816.56923	15816.17613	0.393103288	2.48539E-05	40235.1487
520 T - 606 M	77425.47899	77425.5271	-0.048104192	-6.21297E-07	-1609537.042
520 T - 617 M	50907.90604	50907.98532	-0.079284413	-1.55741E-06	-642092.2381
520 T - 531 B	9758.93026	9758.738933	0.191327642	1.96054E-05	51006.3791
541 T - 606 M	91238.81753	91238.69411	0.123423558	1.35275E-06	739233.4096
541 T - 617 M	57874.31569	57874.55772	-0.242030371	-4.182E-06	-239120.0554
541 T - 531 B	16480.03889	16479.6385	0.400389802	2.42954E-05	41159.98664
606 M - 617 M	50244.73444	50244.58071	0.153727929	3.05958E-06	326841.9393
606 M - 531 B	84686.5341	84686.28296	0.251137139	2.96549E-06	337212.3075
617 M - 531 B	60641.10814	60640.97841	0.129727982	2.13927E-06	467448.1732

Table (6-3) base lines calculation

according to the previous definitions and the base lines calculation we can define Palestine Grid 1923 as a second order class II .

6.2 Recommendations

- awareness raising for the community to understand the Importance And value of the triangulation points Old and modern, and away from going into the Inability of network which reduces the value among community and the definition that it belongs to the Palestinians and does not belong to the Israeli occupation.
- Conduct statistical research limitation the remaining and valid triangulation points in the Palestinian territories in addition to the accessibility (Israeli Settlements).
- It is necessary to document the coordinates of each point , adopting the electronically archive and to facilitate the researchers' access to this information.
- Follow up working on this project using more triangulation points in addition to points beyond the West Bank borders the same network.
- Helmert transformation expose less residual comparing to the results of the 2D affine transformation and taking this into account Helmert transformation more appropriate .
- It is necessary reliance transformation system from WGS 84 or ITRF to the Palestine Grid 1923 Unite the results of work using different tools and reference networks.
- It is necessary to update the network by adding replacement points for the missing ones in addition to the new points in the areas that indigent.
- Updating of the Palestine Grid 1923 according to the latest frame by taking a modern observation to them and analyze it by PPP to transfer it to any period of time (epoch) by the impact of sub plate of tectonic and evaluate the results in terms of the required level of accuracy and finally make a unified center for the issue of related geodetic information to be available to all researchers.

There is no doubt that world has become a small village due to rapid advances in technology, communications, and satellite monitoring so we should keep up with this development through the establish of permanent network or at least a set of points covering the Palestinian territories like global points (IGS) , which I think is possible.

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