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Surveying with GPS, total station and terresterial laser scaner: a comparative study

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Master of Science Thesis in Geodesy No. 3131

TRITA-GIT EX 14-001

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Stockholm, Sweden

May 2014

Abstract

Today, advanced GPS receivers are improving the accuracy of positioning information, but in critical locations such as urban areas, the satellite availability is limited above all due to the signal blocking problem, which degrade the required accuracy. For this reason, different methods of measurement should be used.

The objective of this thesis is to evaluate and compare precision, accuracy and time expenditure of total station (TS), Global Positioning System (GPS) and terrestrial laser scaner (TLS). Comparing precision, accuracy and the required time of these three measurements will improve the knowledge about how much precision and accuracy can be achieved and at what time expense. To investigate this task, a reference network consisted of 14 control points has been measured five times with Leica 1201 TS and served as a reference value for comparison with RTK and TLS measurements. The reference network points were also measured five times with the GPS RTK method so as to compare accuracy, precision and time expenditure with that of TS. In addition, in order to compare the accuracy, precision and time expense of total station and TLS, the North Eastern façade of the L building at KTH campus in Stockholm, Sweden has been scaned five times with HDS 2500 scaner on six target points. These six target points were also measured five times with TS. Then comparison made to evaluate the quality of the coordinates of the target points determined with both measurements. The data were processed in Cyclone, Geo Professional School and Leica geo office software.

According to the result obtained, the reference network points measured with TS were determined with 1 mm precision for both horizontal and vertical coordinates. When using RTK method on the same reference network points, 9 mm in horizontal and 1.5 cm accuracy in vertical coordinates has been achieved. The RTK measurements, which were measured five times, determined with a maximum standard deviation of 8 mm (point I) and 1.5 cm (point A) for horizontal and vertical coordinates respectively. The precision of the remaining control points is below these levels.

The coordinates of the six target points measured with TS on the L building façade were determined with a standard deviation of 8 mm for horizontal and 4 mm for vertical coordinates. When using TLS for the same target points, 2mm accuracy has been achieved for both horizontal and vertical coordinates. The TLS measurements, which were measured five times, determined with a maximum standard deviation of 1.6 cm (point WM3) and 1.2 cm (point BW11) for horizontal and vertical coordinates respectively. The precision of the remaining control points is below these levels.

With regard to time expenditure, it is proved that total station consumed more time than the other two methods (RTK and TLS). TS consumed 82 min more time than RTK but, almost similar time has been consumed by TS and TLS (38 min for TS and 32 min for TLS).

Acknowledgement

This thesis would not be possible without the help of some individuals. First and for most I would like to express my deepest gratitude to my supervisors, Dr. Milan Horemuz and Erick Asenjo for their help, valuable advice, continuing support, endless patience and guidance throughout my thesis. I could learn more than I expected and so I am very lucky to be Milan's student. I would also like to thank my examiner Professor Lars Sjöberg to review my thesis.

I have special thanks to my colleagues who have helped me throughout my thesis work. I appreciate their help during the field measurement since working with laser scaner and total station instrument was very difficult to do alone. They could resist the bad weather condition, snow, which is very harsh for them. I couldn't pass without acknowledging my classmate, Maria, who has contributed a lot to my success.

I am also grateful to Dr. Huaan Fan for his support, encouragement and follow-ups throughout my study. His overall support encouraged me to complete my study properly and on time.

I greatly acknowledge KTH in collaboration with Institute of Land Administration, Bahir Dar University for providing me with this scholarship to study at KTH.

Table of Contents

List of Figures

List of Tables

Acronyms

3D	Three Dimension
BW	Black and White target
ATR	Automatic Target Recognition
CORS	Continuously Operating Reference Station
EDM	Electronic Distance Measurement
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
HDS	High Definition Scanner
IR	Infrared Reflector
P	Point
QC	Quality Control
RL	Reflector-less
RMS	Root Mean Square
RT	Red target
RTK	Real Time Kinematics
SWEPOS	Sweden Positioning System
SWEREF 99	Swedish Reference Frame 99
TLS	Terrestrial Laser Scanning
TPS	Terrestrial Positioning System
UTM	Universal Transverse Mercator
WM	Window Mirror corner

Glossary

- ✓ *Accuracy*: refers to how closely a measurement or observation comes to measure a true or established value
- ✓ *Adjustment*: the process of correcting errors made during the measurement.
- ✓ *Control network*: is a reference that can be served as a reference value for RTK and TLS measurements in order to evaluate the accuracy.
- ✓ *GPS*: Global positioning system is also a surveying instrument that determines coordinates of a point relative to WGS 84. Its height reference is the ellipsoid.
- ✓ *Precision*: refers to how closely repeated measurements or observations come to duplicate the measured or observed values.
- ✓ ScanWorld: is a term used in Cyclone software to refer a scaned scene from one position of the scaner.
- ✓ *Terrestrial Laser Scaning*: can be defined as use of a laser to collect dimensional data of objects in the form of a point cloud.
- ✓ *Time expenditure*: defined as time consumed to perform the required task.
- ✓ *Total station*: is a surveying instrument that determines coordinate of a point indirectly from measured angles and distances. Its height reference is the geoid.
- ✓ *3D quality*: is a measure of accuracy, in which, it is calculated using standard deviations of the 3D coordinates with Eq. (4.3).
- ✓ *Effective time*: is the time needed to measure the required tasks without considering the delayed time (time consumed for changing battery, transporting instruments, etc.)

1 INTRODUCTION

1.1 Background of the study

The research deals with evaluation and comparison of precision, accuracy and time expenditure of three surveying methods. These methods are total station (TS), Global positioning system (GPS), and terrestrial laser scaner (TLS).

Surveying has been an essential element in the development of the human environment for so many centuries. It is an imperative requirement in the planning and execution of nearly every form of construction. Surveying was essential at the dawn of history, and some of the most significant scientific discoveries could never have been implemented, were it not for the contribution of surveying. Its principal modern uses are in the fields of transportation, construction building, apportionment of land, and detail mapping[1].

In surveying, specifically in the area of engineering projects, more sophisticated instruments are employed (total station, laser scaner and GPS) to improve the efficiency and accuracy. Individual surveying techniques has been commonly used in the history of surveying area to collect data from field measurements for various applications with different accuracy capabilities and requirements. The significant development of surveying techniques enabled surveying professionals to evaluate precision and accuracy of different surveying techniques. As a result of this evaluation, many advantages has been gained; basically such as improving the efficiency and accuracy of the results. The accuracy of surveying measurements can be improved almost indefinitely with increased cost (time, effort and money).

Today, the role of surveying got much attention to be used in many applications with better accuracy. The term accuracy is common in many applications to express the quality of observations, measurements or/and calculations.

The required accuracy depends on the needed deliverable output. Applications such as general navigation tasks on the sea, research in oceanography, position and velocity in small scale geophysical exploration are required low accuracy, applications such as hydrography, calibration of transponder system, precise navigation and seismic survey, precise navigation in coastal waters etc. are grouped as medium accuracy requirements and applications which require high accuracy are; precise hydrographic surveying, support of coastal engineering marine, geodynamics, precise continuous height control, engineering construction projects (Sjöberg, 2012).

Accuracy and precision for those in the surveying profession (as well as other technical and scientific fields) are defined in different way. Accuracy refers to how closely a measurement or observation comes to measure a true or established value, since measurements and observations

[1] http://www.britannica.com/EBchecked/topic/575433/surveying [Accessed 09 February 2013]

are always subject to errors. Precision refers to how closely repeated measurements or observations come to duplicate the measured or observed values.

Accuracy of surveying techniques using instruments such as GPS, TS and TLS are dependent on a number of parameters that limit their measurement quality. For instance: multipath, the inherent satellite signal accuracy, signal transmission delay, receiver hardware and software limitations, satellite signal obstruction are some of the problems associated with GPS measurement. On the other hand, limitations stemming from total station are; computed coordinates are in local or target coordinate system, the reference surface for measuring height is geoid. Because of earth's curvature, the accuracy of TS measurement can also be affected by distance limit (the accuracy will decrease when increasing the distance). Finally, accuracy of laser scaner depends on the angle of sight and distance from the object to be scaned i.e. scaning perpendicular to the object is more accurate than slightly inclined scaning. Even if laser scaner can capture thousands of points per second, all of these points cannot be handled easily to manipulate and store.

Therefore, each method has its own advantages and disadvantages. In addition to the above differences, the methods have also different time consumption to do the required tasks. Thus, the scope of the research is to evaluate and compare accuracy, precision and time expenditure of the above three methods.

1.2 Problem statement

Surveying is the technique and science of accurately determining three-dimensional position of points and the distances and angles between them. Various surveying methods (GPS, laser scaner, total station, etc.) are in use. In this research only these instruments have been used.

The latest geodetic GPS receivers are improving the accuracy of positioning information, but in critical locations such as urban areas, the satellite availability is difficult due to the signal blocking problem, multipath etc. which degrade the required accuracy. 3D laser scaners generate up to thousands of points per second, however, handling and manipulating the huge amount of point data is a major problem. To avoid these problems, it is very important to reduce the amount of acquired point data. As a result of this reduction of data, accuracy of the final result will be altered. Total station can measure a single point coordinate precisely, but the computed coordinates are in local or target coordinate system, which needs datum transformation. The accuracy is affected with angle and distance of sight, weather condition, etc.

Considering those limitations, the research will evaluate and compare accuracy, precision and time expenditure of these three surveying methods (total station, GPS and laser scaner).

1.3 Objective of the research

The general objective of this research is intended to evaluate and compare the accuracy, precision and cost (time expenditure) of three methods, i.e. GPS, total station and laser scaning.

Specifically the research intends to:

- Determine and evaluate precision of the reference network which can be served as a reference value for comparison with RTK and TLS
- Determine and evaluate accuracy and precision of GPS RTK and TLS methods
- Determine the cost (time expenditure) of the three methods
- Compare results of the methods based on RMS and standard deviation analysis
- Forward possible recommendations that can improve the precision and accuracy of the three measurement methods

1.4 Significance of the study

This research can be used as a spring board for further studies for those who are interested in the area. On the other hand, the study can help users to choose appropriate methods for a given task. Moreover, since coordinates of the reference points are determined with high precision, it can be serves as a reference values for other users.

1.5 Scope and limitation of the study

The scope of this study is limited within evaluating and comparing the accuracy, precision and time expenditure of three surveying methods. Determining and evaluating the accuracy of the measurement need quite stable weather condition and carefulness. During this work there have been a lot of limitations especially related with whether condition (cold, snow and wind). Due to this problem, the study couldn’t complete according to the time frame work.

1.6 Thesis outline

Chapter one introduces the overall background, problem and objective of the thesis. Chapter two starts with literature review, which describes the overview and fundamentals of GPS, total station and laser scaner. It also presents other’s related work. Chapter three introduces methodology of the thesis and procedures. Chapters four presents the result and discuss the result in detail. Chapter five gives conclusion and recommendation that can improve the result.

2 LITRATURE REVIEW

This section describes some of what others have done in related work in order to give brief idea about the overall concept of precision, accuracy and time expenditure of total station, GPS and TLS.

According to the work by Ehsani *et al*, (2004), a 50 ha area was surveyed with RTK-GPS. The base station and four reference points were established over the highest point in the survey area Corrected GPS signals are transmitted in real time from a base receiver at a known location to one or more rover receivers. Results from RTK GPS method, a horizontal coordinate accuracy of 1 cm has been achieved by compensating for atmospheric delay, orbital errors and other variables in GPS geometry. Comparing this thesis with the above work, 8 mm horizontal coordinate accuracy achieved using the same method (RTK).

According to Lin, (2004), accuracy test was made between GPS RTK and total station. The results showed that a positional accuracy of 14 mm has been achieved using GPS RTK while using total station it was possible to determine 16 mm positional accuracy.

Any blockage from natural or man-made obstacles such as trees and buildings can make use of RTK method limited or impossible. In such cases, total stations are used. Borgelt *et al*, (1996) compared the accuracy of RTK with total station on the free area and they reported a standard deviation of 12 cm in a vertical position with RTK. But in the case of total station, better results (below 5 mm) have been achieved.

Pflipsen, (2006), has tested accuracy and time expenditure of total station versus laser scaner on a pile of sand for comparison purpose. The pile was surveyed twice: once with a laser scaner (Leica HDS 2500) and once with a total station (Leica TS1200), and he processed the data in Cyclone and Geo software respectively. His result showed that almost similar horizontal and vertical coordinate accuracy have been achieved below 9 mm in both methods. The time consumed for the measurements was a little bit more (7 minute) for the total station.

According to the studies conducted by Jonsson, *et al* (2003), RTK measurement was applied to test accuracy of different GPS instruments (Leica, Topcon and Trimble). A network of nine control points was established using total station. Then, the authors performed RTK measurement on the same network and compared results with different instrument. Results obtained from RTK measurement have shown a horizontal and vertical accuracy of 10 mm and 2 cm respectively. When comparing this result with the result of the thesis, better accuracy was achieved in both horizontal and vertical coordinates.

In order to check the compatibility of the RTK method with that of total station method, Ahmed, (2012) tested RTK and total station measurements on an existing network. The objective of the test was to assess the RTK achievable accuracy, to check the repeatability of the results under different satellite configurations and to evaluate RTK performance in urban area. In the test, accuracy and repeatability assessment of the RTK was carried out by comparing the coordinates of points with that of independently precisely determined using a total station. According to the result, the difference between the coordinates of total station and RTK was 2 cm for the horizontal and 3 cm for the vertical coordinates. In comparison with the results of this thesis, the coordinate difference between total station and RTK (coordinates of RTK- coordinates of TS) was 1.8 cm for both horizontal and vertical coordinates.

In another study by Fregonese, et al, (2007), the objective of the study was to access the feasibility of monitoring deformations of large concrete dams using terrestrial laser scaning. For this purpose a test field has been established on the specific dam. First the author established a geodetic network as a reference by Leica TS, and then, using a number of targets on the dam, measurements were taken with a total station and a laser scaner. The reference network was determined with 2 mm horizontal and 3 mm vertical coordinate precision. Targets, mounted on the dam, were measured precisely with a total station, and 3 mm for the horizontal and 4.5 mm for the vertical coordinate accuracy (RMS) has been achieved. On the other hand, using a laser scaner (HDS 300), 4 mm for the horizontal and 8 mm for the vertical coordinate accuracy (RMS) was achieved.

3 OVERVIEW OF SURVEYING METHODS

3.1 Laser Scaning Overview

Laser scaning has been conceived as a method to directly and accurately capture object surfaces. According to Fazlay, (2003), although 30 years old, the commercial market for laser scaning has only developed significantly after 1996. Laser scaning is a method where a surface is sampled or scaned using laser technology. It collects data on the object's shape and possibly its appearance. The collected data can then be used to construct digital, two-dimensional drawings or three-dimensional models useful for a wide variety of applications. The advantage of laser scaning is the fact that it can record huge numbers of points with high accuracy in a relatively short period of time. It is like taking a photograph with depth information. Laser scaners are line-of-sight instruments, so to ensure complete coverage of a structure multiple scan positions are required (Quintero *et al*, 2008).

Fig.3.1: picture ofHDS 2500

In this thesis Leica HDS 2500 scaner (Fig.3.1: picture ofHDS 2500) which has a maximum 40° x 40° field of view was used. With a single point range accuracy of +/- 4 mm, angular accuracies of +/- 60 micro-radians, and a beam spot size of only 6 mm from 0-50 m range, the HDS 2500 delivers survey grade accuracy while providing a versatile platform for data capture. Its 360° x 195° pan and tilt mount and dual internal rotating mirrors enable it to be deployed in virtually any orientation. The combination of high accuracy and field versatility makes the HDS 2500 ideal for fixed or raised installation when leveled tripod mounting is not practical, or for applications with less stringent field of view requirements[2].

Classification of laser scaners (Table 3.1) based on technical specification and measurement principle:

- scaning speed, sampling rate of laser measurement system
- field of view (camera view, profiling, imaging)
- spatial resolution, i.e. number of points scaned in field of view
- accuracies of range measurement system and deflection system

[2] http://hds.leicageosystems.com/en/5940.htm [Retrieved on March 20, 2013]

Table 3.1: Classifications and accuracy of laser scaners based on measurement principle

Measurement technology	Range [m]	Accuracy [mm]	Manufacturers
Time of flight	< 100	< 10	Leica, Mensi, Optech, Riegl, Callidus
	< 1000	< 20	Optech, Riegl
Phase measurement	< 100	< 10	IQSun, Leica, VisImage, Zoller+Fröhlich
Optical triangulation	< 5	< 1	Mensi, Minolta

3.1.1 Registration and Geo-referencing

Registration is the process of integrating the ScanWorlds into a single coordinate system. Here the term ScanWorld is used in Cyclone software to refer to a scaned scene from one station setup. The scaned scene is a collection of 3D points which can be called as point clouds. The registration is derived by using a system of constraints, which are pairs of equivalent or overlapping objects that exist in two ScanWorlds. The registration process computes the optimal overall alignment transformations for each component (Easting, Northing and height) of ScanWorld in the registration, such that the constraints are matched as closely as possible. Combining several datasets into a global consistent model is usually performed using registration. The key idea is to identify corresponding points between the scaned scenes and find a transformation that minimizes the distance between corresponding points. Registration of point clouds in the same coordinate system is the most important step in the processing of terrestrial laser scaner measurements. In order to perform the registration, ScanWorlds have to be overlapped at least 30% each other.

Data points in a captured dataset from any acquisition system may be associated with specific reference coordinate system on the earth's surface. This leads to the term geo-referencing, which can be defined as "the assignment of coordinates of an absolute geographic reference system to a geographic feature"[3].

The ScanWorlds coordinate system is based on the scaner's default coordinate system, unless the scaner was set over known points and these points were imported into ScanControl.

Geo-referencing of scaned data can be defined as a process of transforming the 3D coordinate vector of the laser sensor frame (S-frame) to the 3D coordinate vector of a mapping frame (m-frame) in which the results are required. The m-frame can be any earth-fixed coordinate system such as curvilinear geodetic coordinates (latitude, longitude, and height), UTM, or 3TM coordinates (Charles *et al*, 2009).

[3] http://www.anzlic.org.au/glossary_terms.html [Retrieved March 23, 2013]

To geo-reference a given scene, one first needs to establish control points, input the known geographic coordinates of these control points, e.g. total station measurement, choose the coordinate system and other projection parameters and then minimize residuals. Residuals are the differences between the actual coordinates of the control points and the coordinates predicted by the geographic model created using the control points. They provide a method of determining the level of accuracy of the geo-referencing process.

3.2 Overview of Total Station

In this thesis Leica 1201 total station (see Fig.3.2) was used. The total station is a surveying instrument that combines the angle measuring capabilities of theodolite with an electronic distance measurement (EDM) to determine horizontal angle, vertical angle and slope distance to the particular point.

Fig.3.2: Leica 1201 Total Station

Coordinates of an unknown point relative to a known coordinate can be determined using the total station as long as a direct line of sight can be established between the two points. Angles and distances are measured from the total station to points under survey, and the coordinates (X, Y, and Z or northing, easting and elevation) of surveyed points relative to the total station position are calculated using trigonometry and triangulation. To determine an absolute location, a total station requires line of sight observations and must be set up over a known point or with line of sight to two or more points with known location[4].

Total stations can be manually adjusted or have motors that drive their telescopes very accurately. The most sophisticated total stations can be operated remotely and continuously at various levels of automation.

[4] http://en.wikipedia.org/wiki/Total_station [Retrieved on March 18, 2013]

According to Leica geosystem recommendation[5], in order to get accurate and precise measurements in the daily work, it is important:

- To check and adjust the instrument from time to time.
- To take high precision measurements during the check and adjust procedures.
- To measure targets in two faces. Some of the instrument errors are eliminated by averaging the angles from both faces.

When measurements are being made using the laser EDM, the results may be influenced by objects passing between the EDM and the target. For example, if the intended target is the surface of a road, but a vehicle passes between the total station and the target surface, the result is the distance to the vehicle, not to the road surface.

Instruments equipped with an ATR (Automatic Target Recognition) sensor permit automatic angle and distance measurements to prisms. The prism is sighted with the optical sight. After initiating a distance measurement, the instrument sights and centers the prism automatically. Vertical and horizontal angles and slope distance are measured to the center of the prism and coordinates of the target calculated automatically.

Using Leica 1200+ instruments, the operator does not have to look through the telescope to align the prism or a target because of the ATR. This has a number of advantages over a manually pointed system, since a motorized total station can aim and point quicker, and achieve better precision (Leica 1200+ TS manual).

3.2.1 Measurement accuracy

Total station measurements are affected by changes in temperature, pressure and relative humidity, but it can be corrected for atmospheric effects by inputting changes in temperature, pressure and relative humidity. Shock and stress result in deviations of the correct measurement as a result decreases the measurement accuracy. Beam interruptions, severe heat shimmer and moving objects within the beam path can also result in deviations of the specified accuracy by the manufacture as specified in Table 3.2. It is therefore important to check and adjust the instrument before measurement.

The accuracy with which the position of a prism can be determined with Automatic Target Recognition (ATR) depends on several factors such as internal ATR accuracy, instrument angle accuracy, prism type, selected EDM measuring program and the external measuring conditions. The ATR has a basic standard deviation level of ± 1 mm but above a certain distance, the instrument angle accuracy predominates and takes over the standard deviation of the ATR manual. Leica 1201 total station instruments have standard deviation of 0.3 mgon in both angles which affect the quality of measurement (Leica 1200+ TPS manual). Typical Leica 1200+ instrument accuracy (horizontal and vertical angles) stated by the manuafacturer are given in the Table 3.2.

[5] http://hds.leicageosystems.com/en/5940.htm [Retrieved on March 20, 2013]

Table 3.2: Angle measurement accuracy

Type of instrumnt	Standared devation (Horizontal and Vertical angles)	
	[arcsecond]	[mgon]
1201+	1	0.3
1202+	2	0.6
1203+	3	1.0
1205+	5	1.5

Using different prisms other than the intended prism may cause also deviations and therefore it is important to use a Leica circular prism as the intended target.

3.2.2 Measurement Errors

Some errors, those associated with the instrument, can be eliminated or at least reduced with two face measurement. Table 3.3 shows instrumental errors which influence both horizontal and vertical angles, and their adjustment method.

Table 3.3: Angle errors and their adjustment.

Instrument error	Affects Hz angle	Affects V angle	Eliminated with two face measurement	Corrected with instrument calibration
Line of sight error	Yes	No	Yes	Yes
Tilting axis error	Yes	Yes	Yes	Yes
Compensator errors	Yes	Yes	No	Yes
V-index error	Yes	Yes	Yes	Yes

Collimation axis error (line of sight error) affects the horizontal angle to be deviated and resulting in poor accuracy measurement. This axial error is caused when the line of sight (see Fig.3.3) is not perpendicular to the tilting axis. It affects all horizontal circle readings and increases with steep sightings, but this effect can be corrected by taking average of two face measurement in two rounds. For single face measurements, an on-board calibration function is used to determine collimation errors, the deviation between the actual line of sight and a line perpendicular to the tilting axis.

Vertical axis error (tilting axis error) errors occur when the titling axis of the total station is not perpendicular to its vertical axis. This has no effect on sightings taken when the telescope is horizontal, but introduces errors into horizontal circle readings when the telescope is tilted, especially for steep sightings. As with horizontal collimation error, this error is eliminated by two face measurements.

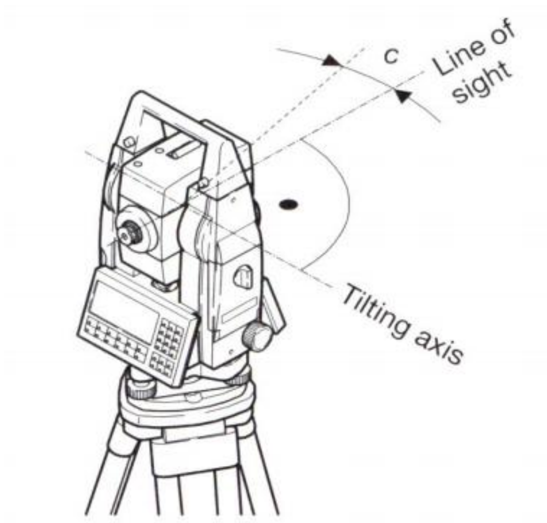

Fig.3.3: Collimation errors

Compensator index error: errors caused by not leveling a theodolite or total station carefully and then cannot be eliminated by taking two face measurements. If the total station is fitted with a compensator it will measure residual tilts of the instrument and will apply corrections to the horizontal and vertical angles for these.

Vertical Collimation (vertical index) error: a vertical collimation error occurs if the 0^o to 180^o line in the vertical circle does not coincide with the vertical axis. This zero point error is present in all vertical circle readings and like the horizontal collimation error it is eliminated by taking two face measurements.

3.2.3 Mode of distance measurement

Measuring with reflector (IR mode)

EDM instruments send a light wave to a reflector and by measuring the phase difference required in returning the reflected light wave to its source, it computes the distance. Using TS 1201 the shortest measuring distance is 1.5 m. but, below this limit, there is no possibility to measure. The specified ranges of different prisms presented in Table 3.4.

Table 3.4: Range limit based on atmospheric condition

Reflector	Range A [m]	Range B [m]	Range C [m]
Standard prism	1800	3000	3500
360^0 prism (GRZ4, GRZ 122)	800	1500	2000
360^0 Mini prism (GRZ 101)	450	800	1000
Mini prism (GMP101)	800	2600	3300

Three sets of atmospheric conditions:

A: Strong haze, visibility 5 km; or strong sunlight, severe heat shimmer

B: Light haze, visibility about 20 km; or moderate sunlight, slight heat shimmer
C: Overcast, no haze, visibility about 40 km; no heat shimmer

Accuracy of standard prism distance measurement depends on the type of reflector and the measuring mode used as indicated in Table 3.5.

Table 3.5: Accuracy of measurements to standard prism

EDM measuring mode	Std dev. Standard prism	Measremnt time [s]
Standard	1 mm + 1.5 ppm	2.4
Fast	3 mm + 1.5 ppm	0.8

Reflectorless EDM

Distance measurement without reflector (RL mode) is applicable in inaccessible locations such as building corners, busy highways, top of light pole, etc. Table 3.6 shows distance accuracy in RL mode. The accuracy depends on the distance between total station and the target to be measured. The shorter distance the better accuracy.

Table 3.6: Distance accuracy in RL mode

Distance	Standard deviation	Measuring time, typical [s]
< 500 m	2 mm + 2 ppm	3 – 6
> 500 m	4 mm + 2 ppm	3 - 6

Both reflector (IR) and reflector less (RL) mode measurements have their own advantage and disadvantages. Their pros and cons are stated in Table 3.7.

Table 3.7: Comparison of IR and RL mode

IR cons	**IR pros**
A person needed for the reflector	Can be measured longer distances
Inaccurate for inside corner measurements	Faster than reflector less
Measurements are difficult in busy highways, top of buildings, sites under construction	Better precision than reflector less
RL cons	**RL pros**
Good accuracy only for shorter distances	No need person for reflector
Less accurate and slower	Can measure inaccessible locations

3.3 Overview of GPS

A GPS receiver (see Fig.3.4) measures the incoming phase of the satellite signals to millimeter precision. However as the satellite signals propagate from satellites to receivers they pass and are affected by the atmosphere. The atmosphere that influences the incoming signal consists of the ionosphere and troposphere. Disturbance in the atmosphere cause degradation in the accuracy of the observations (GPS 500 user manual).

GPS surveying is a differential method; a baseline is observed and computed between two receivers. When the two receivers observe the same set of satellites simultaneously, most of the atmospheric effects are canceled out. The shorter the baseline is the more these effects will be reduced, as more likely it is that the atmosphere through which the signal passes to the two receivers will be identical.

Fig.3.4: GPS receiver

Baseline precision depends on various factors including the number of satellites tracked, satellite geometry, observation time, ephemeris accuracy, ionospheric disturbance, multi path, resolved ambiguities, etc.

3.3.1 Real Time Kinematics (RTK)

Real time kinematics data collection uses differential GPS corrections broadcast by a base receiver to solve for coordinates at a rover receiver in real time. There are several ways to transmit a correction signal from the base station to mobile station. The most popular way to achieve real-time transmission is radio communication. The accuracy of the resulting range measurement depends on the number of satellites in view, resolved ambiguities, satellite geometry, etc.

RTK mode for geodetic measurements is very fast method for surveying and results are available immediately, no need for additional data processing afterwards since correction are made from the base station during the measurement through radio communication (Kostov, 2011).

3.3.2 Comparison of Total Station and GPS

Despite many advantages, surveying using total stations or GPS has disadvantages. Surveying with a total station, unlike GPS, is not disadvantaged by overhead obstructions but, it is restricted to measurements between inter-visible points. Often control points are located distant to the survey area, and traversing with a total station to propagate the control is a time consuming task.

For this reason, GPS is used to bring control to the survey site through before continuing the survey with a total station in areas that limit the use of GPS. Table 3.8 shows their advantage and disadvantages.

GPS can measure points without any line of sight requirement. Since total stations work on the principle of signal reflection, line of sight must be there between total station and prism reflector. This makes GPS more effective tool for control point establishment. However, GPS cannot be used in areas with lot of trees, high rise buildings because of satellite signal interference[6].

Table 3.8: Comparison of GPS and total station

Total station	GPS
Indirect acquisition of 3D coordinates	Direct acquisition of 3D coordinates
Both horizontal and vertical accuracies are comparable	The horizontal accuracy is better than the vertical accuracy
The accuracy depends on the distance, angle and the used prism	The accuracy depends on the satellite availability, atmospheric effect, satellite geometry, multipath
More precise than GPS	Less precise than total station
Satellite independent	Satellite dependent
Needed inter-visibility between the instrument and the prism	Visibility is not needed
Day time data collection	Day or night time data collection

3.3.3 Comparison of Total Station and Laser Scaner

A laser scaner is a surveying instrument that determines a three dimensional coordinates of a given scene in the form of point cloud. Those point clouds represent the position of an object in 3D. Individual points can be compared with points measured by a total station. Their advantages and disadvantages are presented in Table 3.9.

Table 3.9: Comparison of laser scaner and total station

Laser scanner	Total station
Dense information along homogenous surface	Single measurement (angles and distance) of a point
Day or night data collection	Day time data collection
Direct acquisition of 3D coordinates	Indirect acquisition of 3D coordinates
The vertical accuracy is better than the horizontal accuracy	Both angles have comparable accuracy
The accuracy depends on the angle and distance from the facade	The accuracy depends on the distance, prism used
Heavier to transport	Easier to transport

[6] http://totalstation.org/gps [Retrived on April 15, 2013]

Summary of the three methods presented in Table 3.10 for comparison depending on their advantages and disadvantages

Table 3.10: Conclusion of pros and cons of Total station GPS and Laser scaner

Leica total station 1201	Leica GPS 1201	Leica HDS 2500
Local precision is high (1-2mm range)	Real time GNSS is (1-2cm) horizontally and 2-3cm vertically	Local precision is high, +/-4mm for range and +/- 60 micro radian for angle measurements
Uses accurate distance meters and angle encoders to measure position to a nearby reflector	GNSS is relaying on satellites that are approximately 20,000 km away to compute the rovers' position.	Has a maximum 40° x 40° field-of-view. Its 360° x 195° pan & tilt mount and dual internal rotating mirrors enable it to be deployed in virtually any orientation
Provides local coordinates	Provides global coordinates	Provides local coordinates
Flexibility: used in indoors and outdoors. Its accuracy is not degraded by trees blocking or ionospheric effects.	Used in outdoors GNSS is not limited to the line of sight, not weather dependent, not relay on local land marks	Used in indoors and outdoors. Its accuracy is not degraded with trees but it is weather dependent, doesn't work below -6^0c
Weight: lighter than TLS	Somewhat heavier than TS	Heavier than TS and GPS
Day time data collection	Day or night data collection	Day or night data collection
Indirect acquisition of coordinates	Direct acquisition of coordinates	Direct acquisition of coordinates

3.4 Error analysis

Error is the difference between a measured or calculated and the established value of a quantity. In the case of this thesis the established value is the values determined through reference network that controls the detailed survey.

3.4.1 Measurement Errors

There are three types of errors: systematic errors, gross errors and random errors.
Systematic errors are those errors which follow certain physical or mathematical rules. These kinds of errors are: calibration errors, tension in analogue meters, ambient temperature, etc. Those errors can be corrected by applying correction factors, calibrating instruments and selecting suitable instruments.

In most cases gross errors can be caused by human mistakes such as carelessness. The instrument may be good and may not give any error but still the measurement may go wrong due to the operator. Those errors do not follow any physical or statistical rules. This can be corrected by carefulness during the measurement and two face measurements can also detect gross errors.

Examples of those kinds of errors are: taking wrong readings, wrong recording of instrument or target height, reading with parallax error, etc.

Random errors are errors in measurement that lead to measured values being inconsistent when repeated measurements are performed. Those errors are random and affect the measurements in non-systematic way. Random errors can be caused by instrument errors, human factors, physical environment, etc. and they can be improved when frequency of measurement is increased, i.e., the same parameter is to be measured more often.

Errors in measurements stem from three sources: personal, instrumental, and natural. Personal errors are caused by the physical limitations of the human senses of sight and touch. An example of a personal error is an error in the measured value of a horizontal angle, caused by the inability to hold a range pole perfectly in the direction of the plumb line. Personal errors can be systematic, random or gross errors. Personal systematic errors are caused by an observer tendency to react the same way under the same conditions. When there is no such tendency, the personal errors are considered to be random. When personal mistakes such as; recording 69^o instead of 96^o during measurement are gross errors. Instrumental errors are caused by imperfections in the design, construction, and adjustment of instruments and other equipment. Instruments can be calibrated to overcome these imperfections. Natural errors result from natural physical conditions such as atmospheric pressure, temperature, humidity, gravity, wind, and atmospheric refraction.

3.4.2 Accuracy

Field observations and the resulting measurement are never exact. Any observation can contain various types of errors. Often some of these errors are known and can be eliminated or at least reduced by applying appropriate corrections. However, even after all known errors are eliminated, a measurement will still be in error by some unknown value. To minimize the effect of errors and maximize the accuracy of the final result, the surveyor has to use utmost care in making the observations. However, a measurement is never exact, regardless of the precision of the observations.

Accuracy is the degree of conformity with a standard or accepted value. Accuracy relates to the quality of the result. The standards used to determine accuracy can be:

- An exact known value, such as the sum of the three interior angles of a plane triangle is 180°.
- A value of a conventional unit as defined by a physical representation thereof, such as the international meter.
- A survey determined or established by superior methods and deemed sufficiently near the ideal or true value to be held constant for the control of detail survey.

The accuracy of a field survey depends directly upon the precision of the survey. Therefore, all measurements and results should be quoted in terms that are commensurate with the precision used to attain them. Similarly, all surveys must be performed with a precision that ensures that the desired accuracy is attained. Although they are known to be not exact, established control points are deemed of sufficient accuracy to be the control for all other detail surveys.

3.4.3 Precision

Precision is the ability to repeat the same measurement. It is a measure of the uniformity or reproducibility of the result. Precision is different from accuracy in that it relates repeatability of the measurements made. In short a measurement is precise if it obtains similar results with repeated measurements, while accuracy is the closeness to the established value.

3.4.4 Checking accuracy

It is true that any measurement would not be free from errors. In most cases gross errors may happen in a measurement and therefore the accuracy of the measurement needs to be checked in order to avoid the gross errors. There are a lot of accuracy checking mechanisms, for instance, through two face measurement, adjustment, etc. Using these mechanisms, gross errors can be detected. As Csanyi *et al*, (2007) stated out, small magnitude errors of each individual measurement may affect the quality of the final result by considerable large amount. Therefore, the final result may depend on the quality achieved from each individual measurement.

3.4.5 Quality Control

The term quality control (QC) refers to the efforts and procedures that researchers put in place to ensure the quality and accuracy of data being collected using the methodologies chosen for a particular study (Roe, D., 2008).

Quality control measure verifies the accuracy of the surveyed data by checking its compatibility with an independently surveyed data. For instance: in the comparison of TS and TPS, laser scaner targets were extracted from the range of scaning. The coordinates of the extracted targets are then compared with the independently TS surveyed coordinates using RMS analysis. Thus, the total station measurement controls quality of the TLS extracted points. As per Habib *et al*, (1999), the resulting RMS value is a measure of the external and absolute quality of the scaned derived surface.

4 METHODOLOGIES

4.1 Establishing Reference Network

In order to evaluate the accuracy and precision of the surveyed data, primary it has been established a network of control points which can serve as a reference for comparison with RTK and TLS measurement. The reference network was established fourteen control points using a Leica 1201 total station. To determine the network with high precision, measurements have been taken in two faces with two rounds. Four points of the reference network were also measured with static GPS in order to transform the datum from the local coordinate system to the required coordinate system, SWEREF 99. Thus, this network served as a reference value. The precision of the remaining RTK and TLS measurements were evaluated depending on this reference value.

Therefore, to accomplish the objectives of this project, data were collected from field measurement. The field measurements were taken using three different surveying instruments: - Global Positioning System (GPS), laser scaner (LS) and total station (TS). To eliminate instrumental errors such as line of sight errors, tilting axis errors and vertical index errors (see Table 3.3), two face measurements were taken. Since the coordinates determined with total station are provided in local coordinate system, static GPS measurement was needed to transform the datum to SWEREF 99. Then, precision of the network has been obtained from network adjustment and verified for if there have been gross errors were occurred. Detail measurements (RTK on the network and, TLS and TS on the façade) were taken five times to evaluate the precision of the measurement. Finally, accuracy and precision of the detail measurements were tested by RMS and standard deviation analysis respectively.

4.2 Evaluation of Accuracy and Precision

To evaluate the accuracy and precision of the measurement, RMS and standard deviation of the individual measurements were computed. RMS (root mean square error) is a measure of accuracy of the individual measurement. It can be computed from the deviations between true and measured values. True value of the measured quantity is the value which was determined with significantly higher precision. In this project the coordinates of the reference network were considered as 'true' which is determined in 1mm level. RMS was computed using the following formula:

$$RMS(l) = \sqrt{\sum_{i=1}^{n} \frac{(\hat{l}-l_i)^2}{n}} \qquad (4.1)$$

Where: $\hat{l}$ is the established value, l_i is individual measurement and n is the number of measurements.

Standard deviation is a measure of variations of the repeated measurement, i.e. of the precision of each individual observation. It can be computed from the mean values of the individual measurement and the individual measurement. Standard deviation is computed using the following formula.

$$SD(l) = \sqrt{\sum_{i=1}^{n} \frac{(\bar{l}-l_i)^2}{n-1}} \;, \bar{l} = \sum_{i=1}^{n} \frac{l_i}{n} \qquad (4.2)$$

Where: $\hat{l}$ is true or established value, l_i is individual measurement, $\bar{l}$ is mean value of the measurements and n is number of measurements.

4.2.1 Choosing suitable control points for the network and detail survey

Reconnaissance of the project area was the first step in the establishment of control network and followed by marking fourteen control points which are visible each other. Those control points were also suitable for satellite visibility, because RTK method was needed to compare with the TS control points. The points are marked with nails for sustainability reason. The project area was close to L building in the campus of KTH, Stockholm, Sweden (see Fig 4.1).

4.2.2 Setting up targets for laser scaning

In order to compare the results from total station and laser scaner, 21 target points were chosen at the North Eastern façade of the L building. Six black and white target papers were marked as control points for the registration of ScanWorlds. Those target points were also measured with total station. There are requirements to be fulfilled when choosing black and white targets. As Quintero *et al,* (2008) stated out, not only is the station position important, the positioning of the targets carries equal importance. And so, it is important to note that:

- targets be widely separated;
- targets have different heights;
- as few targets as possible on one single line;

4.2.3 Detail survey

Once the reference network and the targets for detail measurement were established, the next step was taking the detail survey. RTK measurement was taken on the reference network to compare the result with total station measurement, and measurements from laser scaning and total station on the façade of L building were taken and the results were compared. In order to evaluate the precision of the measurements, all control points and targets points were measured five times. During all measurements the time required was recorded for comparison.

Total station

In order to determine and compare accuracy, precision and time expenditure of the this method, the façade of L building with black and white paper targets and corners of windows were surveyed five times with the total station. The data was processed in Geo and then, the obtained coordinates of the facade targets were used as constraint during the registration and geo-referencing processes. And time expenditure was also recorded both for field measurement and processing.

Laser Scaning

The façade of the north eastern of the L building was scaned five times with laser scaner from five different views. The scaned ranges were between 6 m to 9 m. Captured point clouds were registered and geo-referenced with precisely determined total station data. Time expenditure for scaning and processing were recorded, analyzed and presented in Table 5.13.

GPS RTK (Real Time Kinematics)

The RTK method was performed to compare accuracy of the network with total station measurements. Using one known coordinate point (DUB) from the adjusted reference network, RTK was used to measure the remaining 13 points five times with 3D quality reported by the receiver of less than 9mm. This 3D quality describes the accuracy of the GPS measurement. Depending on the satellite availability and other sources of errors that affect the GPS measurement, the magnitude of the 3D quality might be small or large. If there is good satellite geometry (i.e. satellites scattered around the four quadrants), good satellite visibility and other GPS errors are small, the 3D quality will be small otherwise it will be large. The 3D quality (Q_{xyz}) can be computed using the formula below (Eq. 4.3):

$$Q_{xyz} = \sqrt{\sigma_x^2 + \sigma_y^2 + \sigma_z^2} \qquad 4.3$$

Where: σ_i is standard deviation of X, Y and Z coordinates

Results of each method were analyzed and compared in order to evaluate the accuracy, precision and time expenditure.

Project area of the study

The project area is the parking lot close to L building, KTH campus, Stockholm, Sweden (Fig.4.1). First, reconnaissance of the project area has been performed, and followed by establishing a network of 14 control points, which have been used as a reference value for the detail survey. The network has been established using Leica 1201 version total station. Figure 4.1 shows the project area and the reference control points. In the Fig.4.1, points dub1, N1, C1 and H1 were measured also by static GPS.

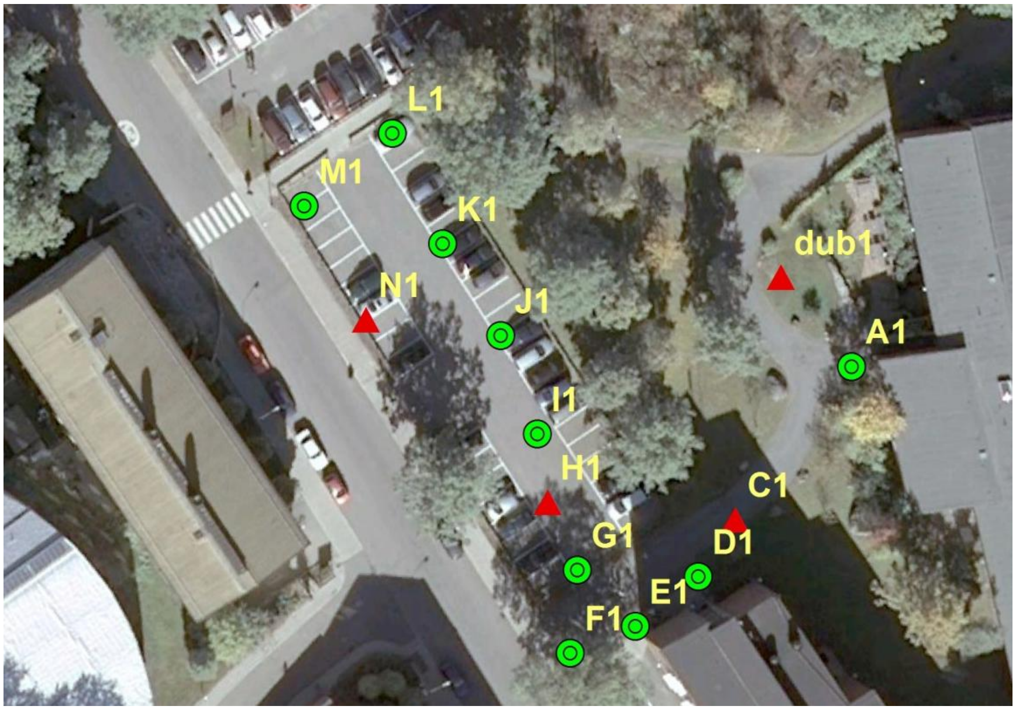

Fig.4.1: Reference Control Points

Data processing

Data were processed in the respective software of the instruments. Data from laser scanning was processed in Cyclone; data from the total station processed in Geo and data from GPS processed in Leica geo office. Registration and geo-referencing of the point cloud was performed to combine ScanWorlds together in one coordinate system of the scanner and then transformed to SWEREF 99.

As a matter of human limitations, imperfect instruments, unfavorable physical conditions and improper measurement routines, which together define the measurement condition, all measurement results most likely contain errors. To reduce the measurement errors on the final results one need to improve the overall condition of the measurement using least square adjustment (Fan, 1997).

Adjustment of the network was performed in Geo software which uses method of least square adjustment. Least square adjustment is a method of estimating values from a set of observations by minimizing the sum of the squares of the differences between the observations and the values to be found.

Least squares method is a classical method which defines the optimal estimate of ***X*** (unknown) by minimizing the sum of the weighted observation residuals squared (Fan, 1997).

$$\sum_{i=1}^{n} P_i \varepsilon_i^2 = minimum \qquad (4.4)$$

$$\boldsymbol{P} = \begin{bmatrix} p_1 & 0 & 0 \\ 0 & p_2 & 0 \\ & \vdots & \\ 0 & 0 & p_n \end{bmatrix} = \boldsymbol{diag(P_1, P_2, \ldots, P_n)}$$

Where p_i: weight of i^th measurement

ε: residual vector and

n : number of observations

In Eq. 4.4 the weight matrix ($\boldsymbol{P}$) is introduced because the network adjustment was a result of distances and angular measurements. In the adjustment process both distances and angles have different weights of a priori standard deviations. Thus, $\boldsymbol{P}$ matrix has been introduced.

Let $\tilde{l}_i \; and \; \varepsilon_i$ represent adjusted value and its residual of observation l_i such that

$\tilde{l}_i = l_i - \varepsilon_i \; (1 \leq i \leq n)$.

Here, $\tilde{\boldsymbol{l}}_{\boldsymbol{i}}$ is a non-linear function of m unknown parameters $x_j (1 \leq j \leq m)$:

$$\tilde{l}_i = f_i(x_1, x_2 \ldots, x_m) \qquad (4.5)$$

Let $x_j^0 \, and \, \delta x_j$ denote an approximate value of x_j and its corresponding correction, such that $x_j = x_j^0 + \delta x_j \; (1 \leq j \leq m)$:

$$\boldsymbol{X} = \begin{bmatrix} x_1 \\ x_2 \\ \vdots \\ x_m \end{bmatrix} = \boldsymbol{X^0} + \boldsymbol{\delta X}, \; \boldsymbol{X^0} = \begin{bmatrix} x_1^0 \\ x_2^0 \\ \vdots \\ x_m^0 \end{bmatrix}, \boldsymbol{\delta X} = \begin{bmatrix} \delta x_1 \\ \delta x_2 \\ \vdots \\ \delta x_m \end{bmatrix} \qquad (4.6)$$

The non-linear equation (4.5) can be expanded by Taylor series and the linearized equation found:

$$\hat{l}_i = l_i - \varepsilon_i = a_{i1}\delta x_1 + a_{i2}\delta x_2 + \cdots + a_{im}\delta x_m + c_i \qquad (4.7)$$

Where: $a_{ij} = \frac{\partial f_i}{\partial x_j}, c_i = f_i(x_1^0, x_2^0, \ldots, x_m^0), i = 1,2, \ldots, n; j = 1,2, \ldots, m$ (4.8)

According to Fan, (1997), the linear system is:

$$\boldsymbol{L} - \boldsymbol{\varepsilon} = \boldsymbol{A} \, \boldsymbol{\delta X} \qquad (4.9)$$

Where:

$$\boldsymbol{L} = \begin{bmatrix} l_1 - c_1 \\ l_2 - c_2 \\ \vdots \\ l_n - c_n \end{bmatrix}, \ \boldsymbol{\varepsilon} = \begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \vdots \\ \varepsilon_n \end{bmatrix}, \ \boldsymbol{A} = \begin{bmatrix} a_{11} & \cdots & a_{1m} \\ \vdots & \ddots & \vdots \\ a_{n1} & \cdots & a_{nm} \end{bmatrix}, \boldsymbol{\delta X} = \begin{bmatrix} \delta x_1 \\ \delta x_2 \\ \vdots \\ \delta x_m \end{bmatrix}, \ \boldsymbol{c_i} = f_i(x_1^0, x_2^0, \dots, x_m^0),$$

$$\boldsymbol{a_{ij}} = \frac{\partial f_i}{\partial x_j} \ (1 \le i \le n; 1 \le j \le m) \tag{4.10}$$

The least square solution of δX can be written as:

$$\boldsymbol{\delta\widehat{X}} = (\boldsymbol{A^T P A})^{-1} \boldsymbol{A^T P L} \tag{4.11}$$

The least squares solutions of the unknown parameter $\boldsymbol{X}$ can be computed as:

$$\boldsymbol{\widehat{X}} = \boldsymbol{X^0} + \boldsymbol{\delta\widehat{X}} \tag{4.12}$$

The least square estimates of the residual, ε is calculated as:

$$\boldsymbol{\hat{\varepsilon}} = \boldsymbol{L} - \boldsymbol{A\delta\widehat{X}} \tag{4.13}$$

Then, the posteriori standard deviation of unit weight is computed as:

$$\boldsymbol{\widehat{\sigma}_o} = \sqrt{\frac{\boldsymbol{\hat{\varepsilon}^T \hat{\varepsilon}}}{\boldsymbol{n-m}}} \tag{4.14}$$

And the cofactor matrix is computed as:

$$\boldsymbol{Q_{\hat{x}\hat{x}}} = (\boldsymbol{A^T P A})^{-1} \tag{4.15}$$

Hence the variance-covariance matrix becomes:

$$\boldsymbol{C_{\hat{x}\hat{x}}} = \boldsymbol{\widehat{\sigma}^2{}_o} \, \boldsymbol{Q_{\hat{x}\hat{x}}} \tag{4.16}$$

And the standard deviation of the unknown parameters can be computed from the diagonal cofactor matrix as:

$$\boldsymbol{\sigma_i} = \boldsymbol{\widehat{\sigma}_o} \sqrt{\boldsymbol{Q_{ii}}} \tag{4.17}$$

Here: $\boldsymbol{Q_{ii}}$ is the diagonal element of cofactor matrix $\boldsymbol{Q_{\hat{x}\hat{x}}}$ in (Eq. 4.15)

Gross errors can be detected with standardized residual using the following formula:

$$\boldsymbol{U_i} = \frac{|\boldsymbol{\hat{\varepsilon}_i}|\sqrt{\boldsymbol{P_i}}}{\boldsymbol{\widehat{\sigma}_o}\sqrt{\boldsymbol{r_i}}} \tag{4.18}$$

Here: $\boldsymbol{P_i}$ is the weight of i^{th} measurement and r_i is the diagonal matrix of $\boldsymbol{R}$:

$$\boldsymbol{R} = \boldsymbol{I} - \boldsymbol{A Q_{\hat{x}\hat{x}} A^T P} \tag{4.19}$$

Here: $\boldsymbol{R}$ is the redundancy and $\boldsymbol{I}$ is the identity matrix

5 RESULTS AND DISCUSSION

5.1 GPS Baseline Processing

Four control points of the reference network were observed with static measurement for three hours. In order to transform the observed points from WGS 84 to SWEREF 99, a baseline was processed from these four control points to SWEPOS station which is Continuously Operating Reference Station (CORS) that provides Global Navigation Satellite System (GNSS). These data are consisting of carrier phase and code range measurements in support of three dimensional positioning. After processing the baselines, the coordinate system was changed in to SWEREF 99. Those coordinates were used as known in the adjustment of the reference network.

The computed coordinates and their standard deviations of the reference network are presented in the Table 5.1.

Table 5.1: Computed coordinates (m)

Point	N	E	H	σN	σE	σH
C	6581703.78	153915.061	58.067	0.001	0.000	0.001
H	6581705.635	153894.633	57.700	0.001	0.000	0.001
N	6581725.636	153874.812	57.523	0.001	0.001	0.000
DUB	6581730.261	153920.049	57.029	0.001	0.000	0.001

5.2 Adjustment

Adjustment is an improvement of the measurement, since measurements are not free from errors. Improvements to observations and coordinates for new points are calculated with various quality measures such as standardized residuals, standard deviation, redundancy numbers, error ellipses etc. The reference network was adjusted first with free adjustment in order to eliminate any contradictions in the fixed points.

5.2.1 Rounds of Measurement

For the sake of eliminating or at least reducing errors emanating from collimation axis error, vertical axis error, compensator errors (longitudinal and transverse), vertical index errors, two face measurements with two rounds have been taken. Mean values of the two face measurements were checked if their differences were below 2 mm for distance and 6 mgon for angles. Atmospheric corrections were also applied before adjustment.

5.2.2 A priori standard deviation

A priori standard deviations have to be considered in the input observation data. Since the measurements consisted of distances and angles, they have different weights to be applied in the adjustment using Eq. (4.4).

These a priori standard deviations are provided by the manufacturers. For the adjustment of the reference network, a priori standard deviation (see Fig.5.1) for the distance was 2 mm + 2 ppm, for the direction 0.6 mgon and standard deviation for the centering error was 1 mm. A priori standard deviation of the height of instrument was 3 mm.

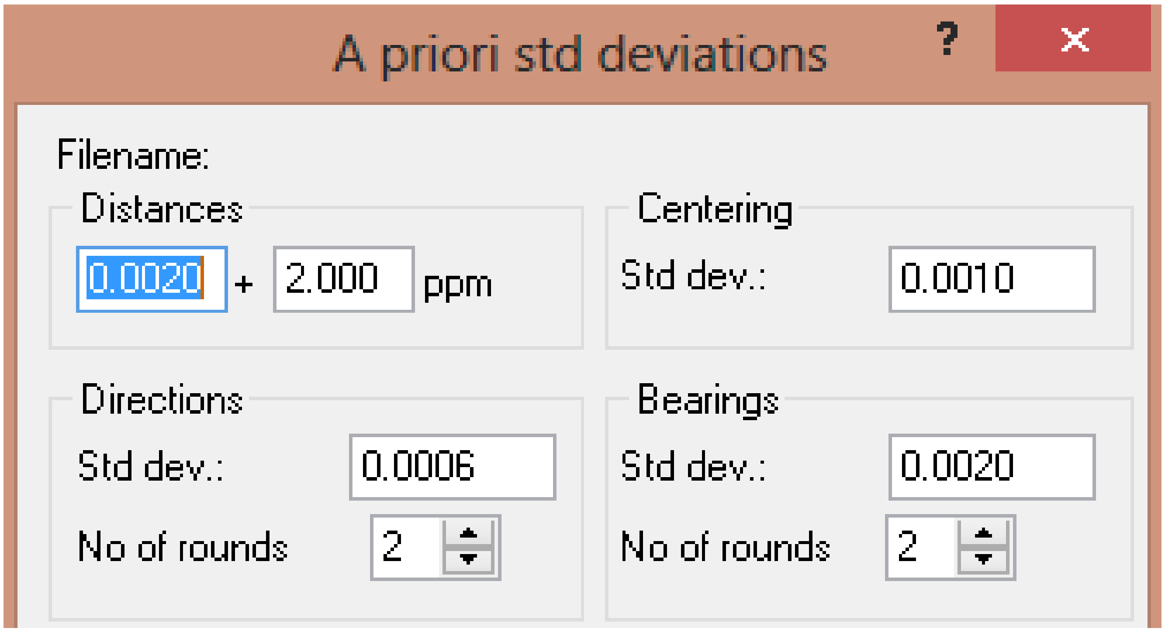

Fig.5.1: A priori standard deviation

5.2.3 Horizontal Network Adjustment

Planimetric coordinates (N and E) were adjusted with free adjustment with translation and rotation. This is a type of adjustment when the reference network is adjusted initially as fully free and is then connected with a transformation. The net fits the known points through the translation in the N and E axes and a rotation. Table 5.2 shows values before adjustment and after adjustment. The residual values computed using Eq. (4.13) show the difference between the adjusted values minus the measured values. The color is controlled by the residual size: Green, if the residual is less than 1 a priori standard deviation. Black, if the residual is less than 2 a priori standard deviations and Red if the residual is greater than 2 a priori standard deviations, in this case the measurement is likely to contain errors. Residuals greater than 2 a priori standard deviations were checked if serious errors had occurred.

Table 5.2: Some of adjusted values and their standard deviations

Obs Type	Station	Object	Value	A pr. StdDev	A pr. SD+C	Residual	Adj. value	StdDev	Std Residual
Direc_n	H1	E1	210.57	0	0.004	-0.006	210.566	0.004	-1.222
Length	H1	E1	16.579	0.005	0.005	-0.005	16.574	0.001	-0.614
Direc_n	H1	G1	223.51	0	0.008	-0.016	223.497	0.009	-1.461
Length	H1	G1	8.08	0.005	0.005	-0.002	8.078	0.001	-0.195
Direc_n	H1	F1	240.23	0	0.004	-0.005	240.227	0.005	-1.192
Length	H1	F1	16.672	0.005	0.005	-0.004	16.669	0.002	-0.408
Direc_n	M1	C1	83.857	0	0.001	-0	83.857	0.001	-0.064
Length	M1	C1	58.17	0.005	0.005	0.007	58.177	0.002	0.831
Direc_n	M1	dub1	53.223	0	0.001	-0.001	53.222	0.001	-0.803
Length	M1	dub1	52.586	0.005	0.005	0.002	52.589	0.002	0.239
Direc_n	M1	H1	100.01	0	0.002	0.001	100.015	0.001	0.188
Length	M1	H1	41.903	0.005	0.005	0.002	41.906	0.002	0.267
Direc_n	M1	N1	111.94	0	0.004	0.008	111.949	0.005	1.359
Length	M1	N1	14.123	0.005	0.005	-0.004	14.119	0.002	-0.449
Direc_n	M1	L1	0	0	0.005	-0.007	399.993	0.007	-1.488
Length	M1	L1	12.486	0.005	0.005	-0.002	12.484	0.001	-0.183
Direc_n	M1	K1	60.647	0	0.004	-0.008	60.639	0.003	-1.278
Length	M1	K1	15.649	0.005	0.005	0.002	15.651	0.001	0.181
Direc_n	M1	A1	61.966	0	0.001	0.003	61.97	0.001	2.241
Length	M1	A1	62.24	0.005	0.005	0.008	62.248	0.002	0.933

As Table 5.2 shows, the maximum and minimum standard deviation of the individual observation of the network was 9 gon in direction and 1 mm in distance respectively. This indicates the measurement was done accurately without gross errors.

Standardized residual values which are computed using Eq. (4.18) are measures of gross errors detected. The colors are also controlled by the standardized residual size. Green: The standard residual is less than 1. Black: The standard residual is less than 2. Red: The standard residual is greater than 2, in which the measurement may be inaccurate and should be checked.

5.2.4 Adjusted coordinates and their standard deviations

Coordinates of all points (Table 5.3) in the reference network were calculated with free adjustment with translation and rotation. First adjusted as fully free and then connected with the X and Y coordinate axes transformation. The net fits the known points through the translation in the X and Y axes and a rotation. Obtained coordinate errors of the network were below 1mm, maximum standard deviations were 0.9 mm in horizontal and 0.7 mm in height. This indicates the network was established with high precision and therefore served as a reference value for the remaining detail measurements. GPS-RTK, laser scaning and total station measurements were evaluated with reference to the established value.

Table 5.3: Adjusted coordinates of the reference network

Point	N	E	H	σN	σE	σH
A	6581720.4857	153927.7596	57.6203	0.0008	0.0006	0.0004
C	6581703.7786	153915.0632	58.2209	0.0007	0.0007	0.0004
D	6581697.5697	153911.0546	58.4491	0.0006	0.0007	0.0004
E	6581692.1084	153904.1971	58.5289	0.0010	0.0008	0.0007
F	6581689.1587	153897.1126	58.6269	0.0012	0.0009	0.0007
G	6581698.2412	153897.8686	57.9827	0.0009	0.0009	0.0007
H	6581705.6416	153894.6336	57.7450	0.0006	0.0006	0.0005
I	6581713.1413	153893.4920	57.5835	0.0009	0.0009	0.0007
J	6581723.8830	153889.5021	57.2779	0.0011	0.0010	0.0007
K	6581733.9285	153883.1438	56.9147	0.0006	0.0007	0.0004
L	6581745.9489	153877.6836	56.5120	0.0010	0.0009	0.0007
M	6581738.0219	153868.0418	56.7599	0.0007	0.0007	0.0004
N	6581725.6336	153874.8107	57.2913	0.0009	0.0010	0.0007
DUB	6581730.2577	153920.0470	57.0608	0.0007	0.0006	0.0004

As it is mentioned in the introduction part, the network consisted of fourteen control points. Fig.5.2 shows the distribution of standard deviation (δ) levels, the colors are controlled by the size of sigma. The measurement's sigma level corresponds directly to the absolute value of the standardized residual. The standardized residual 0-1 gives sigma level 1, 1-2 gives sigma level 2, 2-3 gives sigma level 3 and greater than 3 gives sigma level 3+. Therefore, this sigma level shows quality of the measurement. The least sigma level means the more precise measurement was done and the large sigma level shows the measurement likely to contain gross errors which should be verified and rejected.

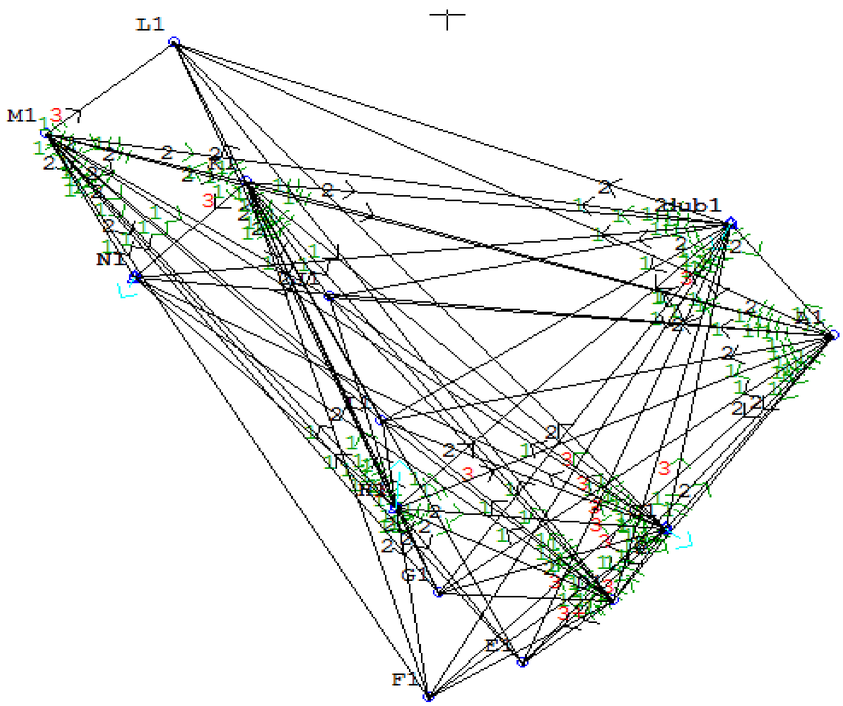

Fig.5.2: Graphical view of the reference network

5.2.5 Vertical Network adjustment

For the vertical network (height) adjustment, data from vertical angles and slope distances has been used. First it was adjusted as fully free and then connected with the transformation. The network fitted the known points through the translation of Z axis. The vertical accuracy was also determined below 1 mm level, the maximum and minimum standard deviations were 0.7 mm and 0.4 mm respectively.

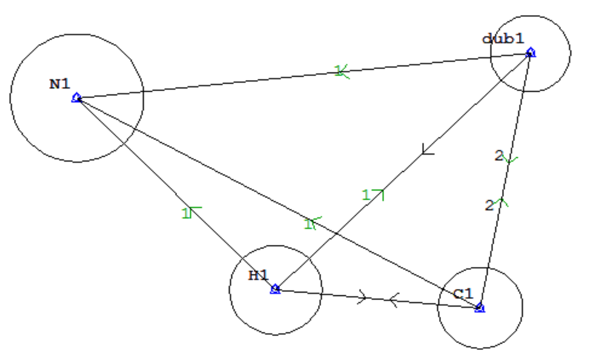

Fig.5.3: Graphical view of height adjustment

5.3 Determination of precision and accuracy of RTK

On the reference network, RTK measurements were taken in order to compare with the total station measurements. Using RTK method, all control points were surveyed five times so as to evaluate the precision of the measurements. To compute the precision of the repeated measurement of the reference network, standard deviation formula Eq. (4.2) has been used. Then, RMS of the RTK measurements were also computed using Eq. (4.1) in order to evaluate how much the measurements were close to the established value.

As the result shows in Table 5.4, the standard deviations are less than 8 mm in horizontal and they reach 1.5 cm in vertical coordinate, which indicates that the repeated measurements were quite close to each other. According to the results obtained by Jonsson *et al* (2003), the standard deviations for the horizontal and vertical coordinate are 9 mm and 2 cm respectively. So, by comparing the author's result with this thesis result, the precisions of the horizontal and vertical coordinate are in mm and cm level respectively.

To evaluate how much RTK measurements were close to the established value, RMS of the RTK measurements were computed (see Table 5.4). This RMS indicates the accuracy of the RTK measurements of the reference network. Accuracy of the horizontal coordinates ranges between maximum 9 mm (points A and D) and minimum 2 mm (point E) and accuracy of the vertical coordinates ranges between maximum 2.2 cm (point M) and minimum 1.1 cm (point I). This result can be compared with the work of Ehsani *et al*, (2004), in which, a horizontal accuracy of 1 cm achieved by compensating for atmospheric delay, orbital errors and other variables in GPS geometry. By comparing the accuracy of horizontal coordinates, they are close to each other. The thesis results are quite reasonable considering the errors attributed from satellite blocking, centering error and so on.

Table 5.4: RTK measurement, its RMS and standard deviations

Point	X	Y	Z	X	Y	Z	σX	σY	σZ
	RTK Mean			RMS			St.D		
A	6581720.487	153927.762	57.616	0.003	0.009	0.012	0.003	0.006	0.015
C	6581703.78	153915.066	58.228	0.007	0.008	0.017	0.007	0.006	0.008
D	6581697.574	153911.055	58.441	0.009	0.005	0.017	0.003	0.004	0.006
E	6581692.109	153904.196	58.521	0.005	0.002	0.015	0.004	0.001	0.005
F	6581689.157	153897.11	58.621	0.004	0.006	0.015	0.003	0.001	0.004
G	6581698.243	153897.867	57.977	0.003	0.004	0.012	0.001	0.003	0.006
H	6581705.645	153894.637	57.738	0.007	0.008	0.015	0.004	0.004	0.007
I	6581713.144	153893.493	57.579	0.007	0.008	0.011	0.005	0.008	0.005
J	6581723.884	153889.504	57.271	0.005	0.007	0.013	0.005	0.005	0.003
K	6581733.927	153883.147	56.907	0.003	0.006	0.015	0.001	0.001	0.002
L	6581745.959	153877.687	56.506	0.008	0.007	0.013	0.007	0.002	0.004
M	6581738.026	153868.045	56.749	0.008	0.007	0.022	0.002	0.003	0.001
N	6581725.636	153874.814	57.285	0.005	0.006	0.012	0.003	0.003	0.006

In order to check if there were significant differences between total station and RTK results, the difference between the total station and the RTK measurements were computed. The difference was computed using the mean values of the measurements. According to the obtained result, it has been proved that there were no significant differences between the two measurements. As indicated in Table 5.5, the maximum difference between TS and RTK is 1 cm in horizontal and 1.1 cm in vertical coordinates. The remaining coordinates are below this level Thus, by comparing the mean coordinates of the RTK measurements with that of total station, their measurement quality has been verified. Therefore, it can be concluded that the measurements were precise and accurate. This can be compared with the standard deviations of the difference between the total station and the RTK measurements.

Table 5.5 The difference between TS and RTK mean

Point	TS Mean			RTK Mean			TS-RTK		
	N	E	H	N	E	H	ΔN	ΔE	ΔH
A	6581720.486	153927.76	57.6203	6581720.487	153927.762	57.616	-0.001	-0.002	0.004
C	6581703.779	153915.063	58.2209	6581703.78	153915.066	58.228	-0.001	-0.003	-0.007
D	6581697.57	153911.055	58.4491	6581697.574	153911.055	58.441	-0.004	0.000	0.008
E	6581692.108	153904.197	58.5289	6581692.109	153904.196	58.521	-0.001	0.001	0.008
F	6581689.159	153897.113	58.6269	6581689.157	153897.11	58.621	0.002	0.003	0.006
G	6581698.241	153897.869	57.9827	6581698.243	153897.867	57.977	-0.002	0.002	0.006
H	6581705.642	153894.634	57.745	6581705.645	153894.637	57.738	-0.003	-0.003	0.007
I	6581713.141	153893.492	57.5835	6581713.144	153893.493	57.579	-0.003	-0.001	0.005
J	6581723.883	153889.502	57.2779	6581723.884	153889.504	57.271	-0.001	-0.002	0.007
K	6581733.929	153883.144	56.9147	6581733.927	153883.147	56.907	0.002	-0.003	0.008
L	6581745.949	153877.684	56.512	6581745.959	153877.687	56.506	-0.010	-0.003	0.006
M	6581738.022	153868.042	56.7599	6581738.026	153868.045	56.749	-0.004	-0.003	0.011
N	6581725.634	153874.811	57.2913	6581725.636	153874.814	57.285	-0.002	-0.003	0.006
DUB	6581730.258	153920.047	57.0608	6581730.258	153920.047	57.061	0.000	0.000	0.000

In addition, the standard deviation of the difference between total station and RTK measurements were calculated using (Eq. 5.2) and compared the result with their coordinate differences. Table 5.6 presents the standard deviation of the difference between total station and RTK. The result shows maximum standard deviation difference of 9 mm horizontally and 1.7 cm vertically. Here the RMS of the total station was not computed because there was no reference value for it.

Table 5.6: Comparison of standard deviations between TS and RTK

	TS			RTK			Diff		
Point	σN	σE	σH	σN	σE	σH	$\sigma_d N$	$\sigma_d E$	$\sigma_d H$
A1	0.001	0.001	0.000	0.007	0.006	0.012	0.007	0.006	0.012
C1	0.001	0.001	0.000	0.009	0.005	0.014	0.009	0.005	0.014
D1	0.001	0.001	0.000	0.007	0.008	0.006	0.007	0.008	0.006
E1	0.001	0.001	0.001	0.008	0.006	0.014	0.008	0.006	0.014
F1	0.001	0.001	0.001	0.007	0.008	0.012	0.007	0.008	0.012
G1	0.001	0.001	0.001	0.005	0.008	0.011	0.005	0.008	0.011
H1	0.001	0.001	0.001	0.005	0.009	0.017	0.005	0.009	0.017
I1	0.001	0.001	0.001	0.009	0.001	0.013	0.009	0.001	0.013
J1	0.001	0.001	0.001	0.009	0.007	0.014	0.009	0.007	0.014
K1	0.001	0.001	0.000	0.005	0.008	0.013	0.005	0.008	0.013
L1	0.001	0.001	0.001	0.005	0.006	0.014	0.005	0.006	0.014
M1	0.001	0.001	0.000	0.007	0.007	0.012	0.007	0.007	0.012
N1	0.001	0.001	0.001	0.004	0.006	0.013	0.004	0.006	0.013

Reliability of the measurements was tested through confidence level. Since the number of measurement is small (n=5), t-distribution was used to compute the confidence interval. t-distribution is a type of probability distribution that resembles normal distribution but for smaller sample size. t-distribution is a bell shaped probability distribution with heavier tail producing values that fall far from the mean. Using 95% confidence level, reliability of the measurements have been verified.

The range of values within which the true value should lie for a given probability is required. This range is called the confidence interval, its bounds called the confidence limits. Confidence limits can be established for that stated probability from the standard deviation for a set of observations. Statistical tables are available for this purpose. A figure of 95% frequently chosen implies that nineteen times out of twenty the true value will lie within the computed limits. The presence of a very large error in a set of normally distributed errors, suggests an occurrence to the contrary and such an observation can be rejected if the residual error is larger than three times the standard deviation.

The confidence interval for the difference between total station (TP) and real time kinematics (RTK) has been calculated as:

$$d_{(RTK-TPS)_i} = P_{TPS_i} - P_{RTK_i} \tag{5.1}$$

$$\sigma_{d_{(RTK-TPS)_i}} = \sqrt{\sigma^2_{TPS_i} + \sigma^2_{RTK_i}} \tag{5.2}$$

$$d_{(RTK-TPS)_i} < k\sigma_{d_{(RTK-TPS)_i}} \tag{5.3}$$

So, the confidence interval is:

$$\pm k\sigma_{d_{(RTK-TPS)_i}} \tag{5.4}$$

Where; $d_{(TPS-RTK)_i}$ is the difference between TPS and RTK coordinates, i = (N, E, H) coordinates

$\sigma_{d_{(RTK-TPS)_i}}$ is the standard deviation of $d_{(TPS-RTK)_i}$, P = coordinate point and k is t-score value calculated from confidence level and degree of freedom.

Table 5.7 presents the confidence interval for the difference between coordinates of RTK and TPS. Since the theoretical difference between their coordinates is zero, the confidence interval is $[-\mathrm{k}\sigma_i, +\mathrm{k}\sigma_i]$, where k is t-score value calculated from confidence level, 95% and degree of freedom (k=2.776). Depending on this requirement, 95% of the points should be inside this confidence interval limit.

Table 5.7: Confidence interval limits and coordinates difference between TPS and RTK

Confidence interval limit and errors difference between TPS and RTK						
	$k\sigma_{d_i}$			Abs[$d_{(RTK-TPS)_i}$]		
Point	**N**	**E**	**H**	**N**	**E**	**H**
A1	0.0196	0.0178	0.0339	0.001	0.006	0.012
C1	0.0251	0.0146	0.0394	0.006	0.010	0.016
D1	0.0195	0.0212	0.0175	0.012	0.000	0.015
E1	0.0218	0.0163	0.0378	0.008	0.001	0.012
F1	0.0197	0.0229	0.0339	0.015	0.005	0.011
G1	0.0147	0.0212	0.0312	0.005	0.001	0.005
H1	0.0129	0.0245	0.0467	0.013	0.003	0.004
I1	0.0240	0.0039	0.0367	0.018	0.007	0.005
J1	0.0241	0.0191	0.0384	0.011	0.002	0.011
K1	0.0140	0.0217	0.0356	0.026	0.023	0.018
L1	0.0142	0.0168	0.0400	0.010	0.003	0.016
M1	0.0195	0.0201	0.0339	0.014	0.006	0.003
N1	0.0119	0.0180	0.0361	0.002	0.003	0.005

From the results calculated, only one point (K) exceeds the confidence interval limit. Therefore, point K should be rejected. The rest points have laid within the confidence interval limit, which accounts 92%.

5.4 Registration and Geo referencing

In order to register the ScanWorlds, five targets (black and white papers) have been used on the façade of L-building. Then using the HDS2500 scaner five ScanWorlds were scaned with more than 80% overlap to each other. As it is described in sub-section 2.1.1, the registration refers merging multiple scans each other in a correct, relative 3D geometry within a single coordinate system. Since those common target points were also measured with the total station five times in two faces, the coordinates have been transformed into SWEREF 99 through geo-referencing. The result shows coordinates of all points were calculated with mean absolute error of constraints less than 9 mm. Finally, the mean values of the coordinates of five ScanWorlds were computed in order to evaluate the precision of the laser scaning measurement. The difference between laser scaner measurement and total station measurement were determined and compared in the same procedure as RTK and TS. After ScanWorlds have been registered, TS measured points were imported so as to transform the scanner's coordinate system in to the required coordinate system, SWEREF 99.

Fig.5.4 shows all geo-referenced points on the façade of L building. For the sake of simplicity understandability, the author used black and white (BW) targets, corners of the windows mirror (WM) and edges of the black and white target papers (P). These coordinates were determined with mean absolute error of less than 9 mm.

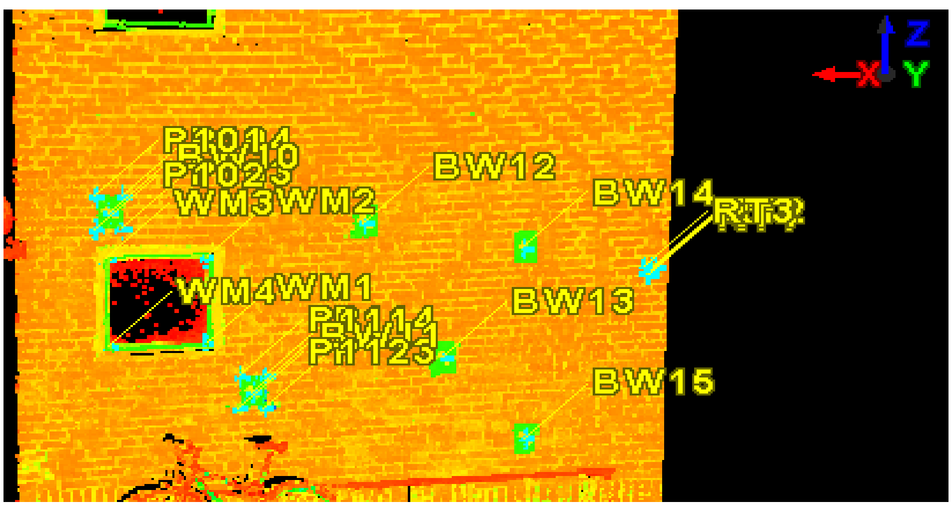

Fig.5.4: Geo-referenced points

In order to eliminate gross errors from the measurement, two big errors (greater than 1 cm level) in the measurement were identified and rejected from the registration Fig.5.5 shows accuracy of registered ScanWorlds. In addition, each error vector of the targets was also checked individually if gross errors had been occurred.

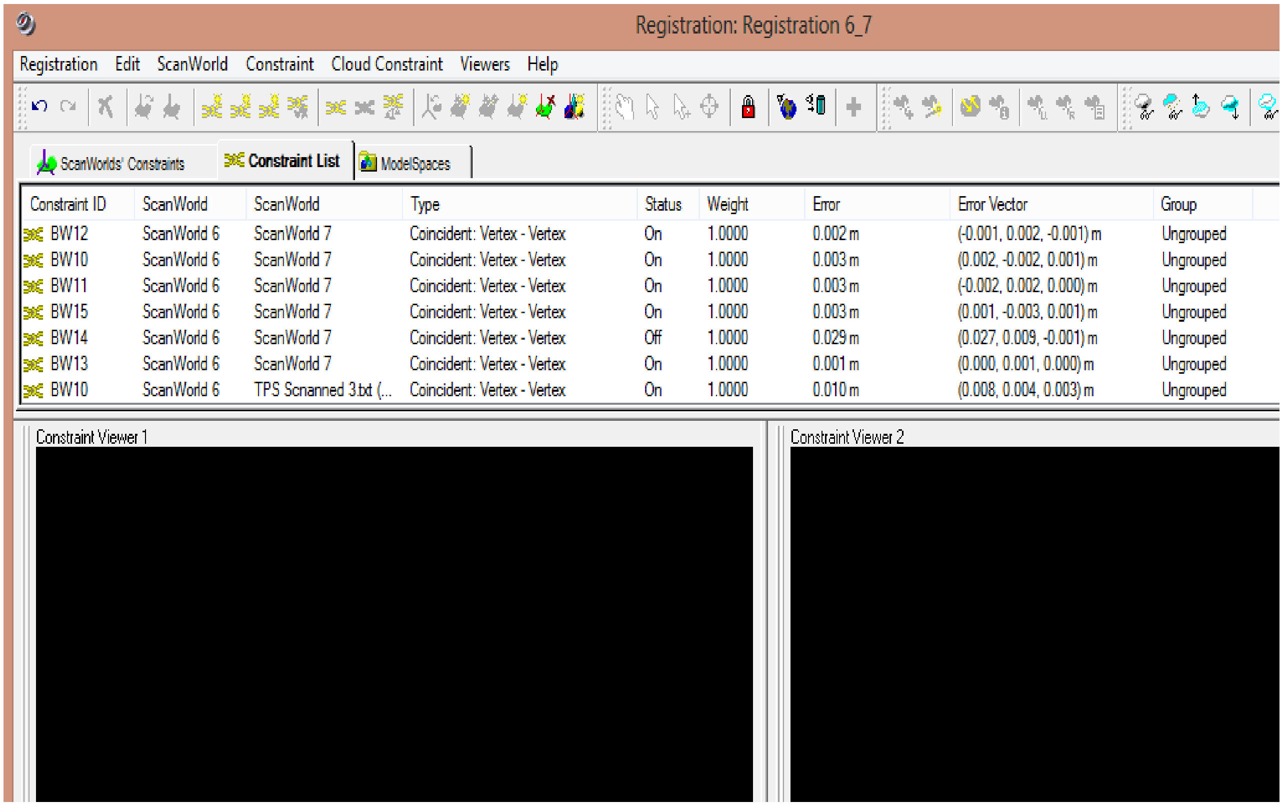

Fig.5.5: Registered and geo-referenced ScanWorlds

5.5 Comparison of Laser Scaner and Total Station result

Results from laser scaner and total station measurements are quite similar in both horizontal and vertical coordinates. As it is described in section 4.5, ScanWorlds were scaned five times to compute the precision of the measurements. Registration needs at least two ScanWorlds at a time so as to provide the ScanWorlds in the same coordinate system. So, by forming five pairs of ScanWorlds from the five measurements, five registrations have been performed using the five black-white targets as constraint. After performing five registrations, standard deviations of the measurement were calculated as it is shown in Table 5. 8. It shows that mm level precision has been achieved for all coordinates.

In the same way, mm level precision has been achieved with the total station also (see Table 5.9). Even if their magnitude is within mm level, still it could possible to achieve better precision with the total station.

Table 5. 8: Standard deviations of the registered TLS

	Extracted points from Terrestrial Laser Scaning					
Point	**N**	**E**	**H**	σN	σE	σH
BW10	6581693.863	153909.234	61.651	0.002	0.005	0.001
BW11	6581693.247	153908.272	60.000	0.005	0.004	0.012
BW12	6581692.756	153907.503	61.529	0.005	0.005	0.002
BW13	6581692.385	153906.925	60.245	0.003	0.006	0.006
BW15	6581691.992	153906.315	59.442	0.002	0.003	0.003
P101	6581693.915	153909.322	61.803	0.004	0.007	0.004
P102	6581693.938	153909.322	61.505	0.009	0.007	0.006
P103	6581693.808	153909.146	61.508	0.006	0.003	0.009
P104	6581693.806	153909.147	61.803	0.006	0.013	0.006
P111	6581693.296	153908.357	60.143	0.006	0.008	0.004
P112	6581693.296	153908.363	59.852	0.006	0.010	0.003
P113	6581693.186	153908.188	59.845	0.004	0.003	0.008
P114	6581693.186	153908.187	60.143	0.006	0.004	0.003
RT1	6581691.413	153905.406	60.974	0.003	0.006	0.005
RT2	6581691.386	153905.364	61.049	0.006	0.007	0.003
RT3	6581691.435	153905.447	61.048	0.006	0.003	0.003
RT4	6581691.415	153905.406	61.005	0.006	0.006	0.011
WM1	6581693.392	153908.594	60.445	0.007	0.008	0.006
WM2	6581693.386	153908.608	61.236	0.004	0.006	0.004
WM3	6581693.794	153909.284	61.225	0.008	0.016	0.006
WM4	6581693.804	153909.266	60.454	0.005	0.004	0.005

In order to compare the results of the laser scaning measurement with that of total station, five measurements were taken with total station on the façade of L-building. The black and white (BW) targets, the edges of these BW targets and the corners of windows mirrors (WM) were measured on the façade. Then the mean values of the measurements have been determined to compute its precision. Table 5.9 presents the mean values of five measurements and their standard deviations. The standard deviations are quite small relative to the laser scaninig measurements.

Table 5.9: Standard deviations of TS measurements

	Total Station points					
	N	E	H	σN	σE	σH
BW10	6581693.861	153909.232	61.652	0.002	0.008	0.001
BW11	6581693.246	153908.270	59.993	0.001	0.001	0.001
BW12	6581692.755	153907.501	61.528	0.004	0.002	0.001
BW13	6581692.387	153906.927	60.246	0.001	0.002	0.001
BW15	6581691.991	153906.316	59.443	0.001	0.003	0.001
P101	6581693.916	153909.320	61.802	0.002	0.001	0.002
P102	6581693.926	153909.320	61.507	0.006	0.001	0.002
P103	6581693.806	153909.146	61.505	0.002	0.003	0.001
P104	6581693.805	153909.144	61.801	0.002	0.002	0.001
P111	6581693.299	153908.359	60.144	0.001	0.001	0.001
P112	6581693.298	153908.365	59.851	0.001	0.002	0.002
P113	6581693.187	153908.188	59.844	0.001	0.003	0.001
P114	6581693.187	153908.186	60.142	0.001	0.003	0.001
RT1	6581691.413	153905.408	60.972	0.003	0.002	0.001
RT2	6581691.387	153905.367	61.049	0.002	0.001	0.001
RT3	6581691.439	153905.447	61.049	0.001	0.003	0.001
RT4	6581691.414	153905.407	61.003	0.006	0.001	0.001
WM1	6581693.394	153908.596	60.448	0.003	0.003	0.003
WM2	6581693.385	153908.606	61.234	0.003	0.003	0.003
WM3	6581693.796	153909.288	61.228	0.003	0.002	0.004
WM4	6581693.805	153909.265	60.453	0.004	0.003	0.003

Table 5.10: Standard deviation and RMS of TLS measurement

	TLS Mean values			RMS			**Std D**		
	N	E	H	N	E	H	σN	σE	σH
BW10	6581693.863	153909.234	61.651	0.002	0.002	0.001	0.002	0.005	0.001
BW11	6581693.247	153908.272	60	0.001	0.002	0.007	0.005	0.004	0.012
BW12	6581692.756	153907.503	61.529	0.001	0.002	0.001	0.005	0.005	0.002
BW13	6581692.385	153906.925	60.245	0.002	0.002	0.001	0.003	0.006	0.006
BW15	6581691.992	153906.315	59.442	0.001	0.001	0.001	0.002	0.003	0.003
P101	6581693.915	153909.322	61.803	0.001	0.002	0.001	0.004	0.007	0.004
P102	6581693.938	153909.322	61.505	0.012	0.002	0.002	0.009	0.007	0.006
P103	6581693.808	153909.146	61.508	0.002	0	0.003	0.006	0.003	0.009
P104	6581693.806	153909.147	61.803	0.001	0.003	0.002	0.006	0.013	0.006
P111	6581693.296	153908.357	60.143	0.003	0.002	0.001	0.006	0.008	0.004
P112	6581693.296	153908.363	59.852	0.002	0.002	0.001	0.006	0.01	0.003
P113	6581693.186	153908.188	59.845	0.001	0	0.001	0.004	0.003	0.008
P114	6581693.186	153908.187	60.143	0.001	0.001	0.001	0.006	0.004	0.003
RT1	6581691.413	153905.406	60.974	0.000	0.002	0.002	0.003	0.006	0.005
RT2	6581691.386	153905.364	61.049	0.001	0.003	0	0.006	0.007	0.003
RT3	6581691.435	153905.447	61.048	0.004	0	0.001	0.006	0.003	0.003
RT4	6581691.415	153905.406	61.005	0.001	0.001	0.002	0.006	0.006	0.011
WM1	6581693.392	153908.594	60.445	0.002	0.002	0.003	0.007	0.008	0.006
WM2	6581693.386	153908.608	61.236	0.001	0.002	0.002	0.004	0.006	0.004
WM3	6581693.794	153909.284	61.225	0.002	0.004	0.003	0.008	0.016	0.006
WM4	6581693.804	153909.266	60.454	0.001	0.001	0.001	0.005	0.004	0.005

Table 5.10 shows the quality of terrestrial laser scaning measurements in the form of precision and accuracy. The accuracy is determined with 2 mm for both horizontal and vertical coordinates.

The difference between the established and measured values theoretically should be zero, which is the center of the t-distribution graph. To calculate the upper and lower interval limit, constant k could be computed from confidence level and degree of freedom. Since the number of measurements is n=5, degree of freedom will be n-1, which is 4. And k was computed in excel using the function [tinv(0.05,4)] with confidence level of 95%. k=2.776

Thus, the confidence interval limit is:

$$\pm k\sigma_{d_{(TLS-TPS)_i}} \qquad (5.5)$$

Where: $\sigma_{d_{(TLS-TPS)_i}}$, is the standard deviation difference between TS and TLS

k: constant
$\pm$: +, for the upper limit and -, for the lower limit intervals

Table 5.11: Confidence interval limit for the difference between TPS and TLS

Confidence interval limit and errors obtained from differences TPS and TLS						
$\sigma_{d_{(TLS-TPS)_i}}$				Abs[$d_{(TLS-TPS)_i}$]		
Point	N	E	Z	N	E	H
BW10	0.034	0.027	0.028	0.002	0.001	0.001
BW11	0.015	0.012	0.034	0.002	0.001	0.007
BW12	0.017	0.015	0.005	0.002	0.002	0.001
BW13	0.009	0.017	0.017	0.001	0.002	0.001
BW15	0.007	0.012	0.010	0.001	0.001	0.001
P101	0.012	0.019	0.014	0.001	0.002	0.002
P102	0.131	0.019	0.017	0.012	0.002	0.001
P103	0.017	0.011	0.025	0.002	0.001	0.003
P104	0.017	0.037	0.016	0.001	0.004	0.002
P111	0.017	0.021	0.011	0.002	0.002	0.001
P112	0.016	0.029	0.010	0.002	0.002	0.001
P113	0.012	0.010	0.022	0.001	0.001	0.001
P114	0.016	0.014	0.010	0.001	0.001	0.001
RT1	0.010	0.017	0.015	0.001	0.001	0.001
RT2	0.018	0.019	0.009	0.001	0.003	0.000
RT3	0.016	0.011	0.009	0.003	0.001	0.001
RT4	0.024	0.018	0.032	0.001	0.001	0.002
WM1	0.020	0.024	0.018	0.002	0.002	0.003
WM2	0.013	0.019	0.012	0.001	0.002	0.002
WM3	0.024	0.044	0.019	0.001	0.003	0.003
WM4	0.017	0.013	0.015	0.001	0.001	0.001

As the result shows in Table 5.11, all values calculated from the coordinate difference between total station and laser scaner lied within the allowable confidence interval limit.

5.6 Comparison of Time Expenditure

In order to compare the cost (time expenditure) of the methods applied, effective time has been recorded throughout the measurements. Effective time refers to the time needed to measure the required tasks without considering the delayed time due to some problems. The specified time is specific to this measurement because it depends on the operator engaged. For the convenient of comparison, time expenditure was classified in to time needed for total station versus GPS, and total station versus laser scanner. The required time does not include the time for transportation of instruments from store to the field and vice versa, and delayed time due to some problems such as: battery problem, incorrect reading, etc.

5.6.1 Total Station versus GPS

As it is described in chapter one, the reference network consists of 14 control points, which were measured from 7 stations towards all 14 points. This was done using two faces with two rounds of measurements. The overall tasks were classified as field work and office lab work. But, here the time consumed was recorded and compared only for the field measurement between TPS and detail RTK on the reference network. Time allocated for every step of the measurement is presented in Table 5.12. Time needed to setup the tripod of the instrument (total station) on one station was recorded and then multiplied by the number of instrument stations to determine the time expended on all instrument setups. In this project, the reference network has consisted of seven instrument stations. The time expended for one setup of a tripod on one target is multiplied by14 to calculate the expended time on tripod setup, since 14 is the number of control points in the reference network. The same is true for the rest of measurement steps to determine the expended time for total station measurement. Thus, the required total time with the total station was 168 min (2 hours and 28 min). Time expended for GPS RTK was recorded as time required for the reference base and for the rover. For the reference station, time was calculated as: time required for tripod setup plus to center it which was 8 min. For the rover measurement, time has been recorded as: time needed to center the rover plus time to record and to change to the next station and then multiplied by the number of control points (13), totally it was 86min (1 hour and 26 min).

Finally, the overall expended time on the reference network using total station and GPS RTK measurement has been recorded and compared. The time needed for the total station measurement was 2 hours and 28 min and that of GPS RTK measurement was 1 hour and 26 min. When comparing required time of the two methods, total station was consumed more time than RTK.

Table 5.12: Time expenditure for TS and RTK measurements for the reference network

Total station			RTK	
Time expenditure in (min)				
Measurement steps	Instrument	Prism	Reference	Rover
Tripod Setup	4	4	5	---
Centering	5	3	3	2
Aiming	3	---	---	---
Recording	2	---	---	2
Changing station	---	4	---	2
Sum	14	11*14	8	6*13
	168		86	

5.6.2 Laser scaning and total station

In this sub section time comparison is presented in Table 5.13 between total station and laser scaning measurement. In this measurement, both methods have shown somewhat comparable time consumption. Time required for the scaning was recorded as: time needed for putting black-white targets plus time for tripod setup plus time for aiming and recording, which was totally 32 min. Using the total station, required time was recorded as: time needed for tripod setup on instrument station and back-sight point plus time for centering, aiming and recording. There were 21 points on the façade including the black-white targets. The required total time using total station was 38 min. When comparing time consumed for those two methods, there was only 6 minute difference to accomplish each task. So, it was possible to measure with the scanner faster than that of total station.

Table 5.13: Time expenditure for TS and laser scaner on the façade

	Façade measurement with total station			Façade measurement with laser scanner	
	Time expenditure in [min]				
	Tasks	Instrument	Targets/ prism	Instrument	Targets
Field work	Putting black-white paper on façade	---	---	---	20
	Tripod Setup	4	4	5	---
	Centering	5	3	---	---
	Aiming	12	---	2	---
	Recording	10	---	5	---
	Changing station	---	---	---	---
	Sum	31	7	12	20
		38		32	

6 CONCLUSION AND RECOMMENDATION

6.1 Conclusion

Today total station (TS), GPS and terrestrial laser scaner (TLS) are used for many tasks within different applications, for example, geodesy, engineering, architectural and mining surveys and documentation of cultural heritage with different accuracy level depending on the needed requirements.

The purpose of this thesis work was to evaluate and compare accuracy, precision and time expenditure of three surveying methods (TS, GPS and TLS). The comparison was made between TS versus GPS RTK on the reference network and TS versus TLS on the façade of L building, KTH campus, Stockholm, Sweden. To accomplish the objectives of the thesis, three major tasks have been performed. 1. A network of 14 control points was established with high precision (l mm) with total station and served as a reference or established value. 2. On the same network, RTK method was performed to compare the result with that of total station. 3. Finally, by scaning the targets on the façade of the L building and measuring the same target points with the total station, comparison has been made between the extracted coordinates of the façade and the coordinates measured by the total station.

In every task of the measurement, time expended was recorded and compared (see Table 5.13 and Table 5.13) separately for TS versus GPS and TS versus TLS methods respectively. Then, in order to evaluate the precision and accuracy of the RTK on the reference network and TLS and TS on the façade measurements were taken five times.

Based on the results obtained, precision of the reference network determined with 1 mm standard deviation both for horizontal and vertical coordinates for all points. This result has been achieved because of the round measurements and two face measurements with the total station. On the same control points of the network, RTK method was performed and according to the result obtained, the standard deviations are less than 8 mm in horizontal and they reach 1.5 cm in vertical coordinate, which indicates that the repeated measurements were quite close to each other. The accuracy of the RTK measurements on the network, which is expressed by RMS, are less than 9 mm in horizontal and they reach 2.2 cm in vertical coordinates.

Precision of the TS measurement on the façade of L building has been determined with maximum standard deviation of 8 mm (point BW10) in horizontal and 4 mm (point WM3) in vertical coordinates. On the same points of the façade, coordinates extracted from the TLS measurement has been determined with maximum standard deviation of 1.6 cm (point WM3) and 1.2 cm (point BW11) in horizontal and vertical coordinates respectively. But the remaining points were below this level. Then the accuracy of the TLS measurements was determined with maximum RMS of 4 mm (point WM3) in horizontal and 7 mm (BW11) in vertical coordinates.

Finally, the time expenditure summarized as more time (82 min) was consumed for TS measurement in the former method, but almost similar time (38 min for TS and 32 min for TLS) was consumed in the latter case.

In order to evaluate the quality of the measurement, absolute value of each coordinate difference between each method should not be exceed $k\sigma_{d_i}$, which limits the errors not to be beyond certain limit by multiplying their sigma differences with constant k (2.776). Based on this quality control measure, more than 95% of the total result has achieved the requirement. This can be interpreted as values which lied within the allowable limit (interval limit), considered as accepted values. But values out of the interval limit considered as risk values, which might contain gross errors. There was one point which was out of the interval limit and was rejected.

Therefore, it can be concluded that there were no gross errors in the measurement; because the measurements were made precisely and accurately. For instance: when measuring using total station, two face measurements was taken to eliminate some errors such as collimation axis errors, tilting axis errors, etc. When using GPS RTK method, small tripod was used to erect the rover vertical. Initially, I expected to achieve accuracy in mm level. But, due to some errors (like centering error, instrumental error, satellite signal obstruction), some results have been deviated in to cm level.

6.2 Recommendations

The obtained results from this thesis will hopefully improve the knowledge about accuracy, precision and time consumption of the three methods used (TS, GPS and TLS). One can differentiate which instrument should be used for which specific application depending on the presented results. For further improvement of accuracy, the following recommendations are forwarded:

- Total station (Leica 1201) should be calibrated at some regular intervals. Since there was problem in the level bubbles; one on the tribrach and the other on the total station couldn’t be leveled at the same time. So, once calibrated the instrument, it will improve the level of accuracy.
- It can be achieved better accuracy by calibrating those instruments before the measurement campaign.
- Applications which require high precision so as to serve as reference value, such as control point establishments, I recommend to use total station instead of GPS.
- It was very difficult to manage the field measurement alone, specially establishing the reference network has been a big problem. There will be a possibility of occurring gross errors and therefore, I recommend working in group.

REFERENCES

Ahmed, E.M. (2012). Performance Analysis of the RTK Technique in an Urban Environment, Australian Surveyor, 45:1, 47-54.

Angelo, J.B. (2004), Integration of Laser Scaning and Close Range Photogrammetry the last decade and beyond, Ottawa, Canada.

ANZLIC (Australia and New Zealand Land Information Council), 2006. Glossary: spatial information related terms. Retrieved on March 23, 2013 from http://www.anzlic.org.au/glossary_terms.html.

Borgelt S C., Harrison J D., Harrison, K. A. Sudduth, and S. J. Birrell (1996). Evaluation of GPS for Applications in Precision Agriculture; Appl. Eng. Agric. 12(6) 633–638.

Clark R.L. and Lee R. (1998). Development of Topographical Maps for Precision Farming with Kinematic GPS; Trans. ASAE. Vol. 41(4) 909–916.

Csanyi, N. and Toth, C. (2007). Improvement of LiDAR Data Accuracy Using LiDAR Specific Ground Targets. Ohio State University, Department of Civil and Environmental Engineering and Geodetic Science, Columbus, USA.

Ehsani, M. R., Upadhyaya, S. K. and Mattson, M. L. (2004). Seed Location Mapping Using RTK GPS; Trans. ASAE. Vol. 47(3): 909-914.

El-Hakim, S.F., and Beraldin, J.A. (2003). Critical Factors and Configurations for Practical Image-Based 3D Modeling. In: 6th Conference on 3D Measurement Techniques. Zurich, Switzerland. Sept. 22-25, Vol. II, pp. 159-167.

Fan, H. (1997). Theory of Errors and Least Squares Adjustment, KTH, Stockholm, Sweden.

Fazlay, F.S. (2003). LIDAR Image Processing Creates Useful Imagery, ArcUser Magazine, University of Mississippi.

Forstner, W. (1994). Diagnostics and Performance Evaluation in Computer Vision. In: NSF/ARPA Workshop: Performance versus Methodology in Computer Vision.

Fraser, C.S. (1987). Limiting Error Propagation in Network Design. Department of Photogrammetric Engineering and Remote Sensing, University of Melbourne, Victoria, Australia.

Fregonese, L., Pael A., van Schadewijk A. (2007). Structural Monitoring of a large Dam by Terrestrial Laser Scaning, University of Milan, Italy.

Kostov, G. (2011). Using of both Fast Static and RTK Modes for GNSS Determinations to Obtain Required high Accuracy and Productivity, According to the Current Possibilities of the IT. Marrakech, Morocco.

Habib, A. and Schenk, T. (1999). A New Approach for Matching Surfaces from Laser Scaners and Optical Sensors, Ohio State University, Department of Civil and Environmental Engineering and Geodetic Science, Columbus, USA.

High Definition Scaner (HDS) Leica Geo-system, retrieved on March 20, 2013 from http://hds.leicageosystems.com/en/5940.htm.

Charles K.T. and Jie S. (2009). Topographic Laser Ranging and Scanning Principles and processing, retrieved on 20 March, 2013 from http://www.asprs.org/a/publications/pers/2009journal/october/review.pdf.

Jonsson K.O., Andersson A., Jacobsson S.O., Vandevoorde S., Lambert D.M., Fowler C.J. (2003), SWEPOS Network-RTK Services, status, applications and experiences. Presented at ION GPS/GNSS 2003, 9-12 September, 2003, Portland, Oregon, U.S.A.

Lin, L.S. (2004). Application of GPS RTK and total station systems on dynamic monitoring land use. Proceedings of the ISPRS Congress Istanbul, Turkey 2004.

Leica 1200+ total station manual.

Leica GPS 500 user manual.

Quintero, M.S., Alonzo A. and Scot R. (2008). Theory and Practice on Terrestrial Laser Scaning Training material based on practical applications, version 4.

Pflipsen, B. (2006). Volume Computation: A Comparison of Total Station versus Laser Scanner. Master thesis, Department of Technology and Built Environment, University of Gävle, Sweden.

Roe, D. (2008). Quality control. In P. Lavrakas (Ed.), *Encyclopedia of survey research methods.* (pp. 650-652). Thousand Oaks, CA: SAGE Publications, Inc. doi: http://dx.doi.org/10.4135/9781412963947.n422.

Satalich, J. and Ricketson, R. (1998). Field test of Trimble 4000 Real-Time Kinematic GPS Survey System; Journal of Surveying Engineering. Vol. 124(1) 40–48.

Sjöberg, L.E. (2012). Lecture notes on the course AH2923 Global Navigation Satellite System (GNSS), Division of Geodesy, KTH, Stockholm, Sweden.

Surveying, Britannica online, retrieved on 09 February, 2013, from http://www.britannica.com/EBchecked/topic/575433/surveying.

Total Station. Wikipedia, the free encyclopedia. Retrieved on March 18, 2013 from http://en.wikipedia.org/wiki/Total_station.

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ISRN KTH/GIT/EX--14/003-SE