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Title: Height Modernization from Static GPS Networks in Oregon: Evaluating NGS Guidelines and OPUS-Projects

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Determining accurate elevations is important for many engineering and scientific applications, and finding these heights via GNSS increases efficiency and significantly reduces the costs as compared to precise geodetic leveling. The National Geodetic Survey (NGS) has published guidelines for determining both ellipsoidal and orthometric heights with GPS to within 2 cm at 95% confidence. However, these guidelines, known as NOS-NGS 58 and 59, were developed based on technologies and experiences in the 1990s. Since then, GNSS-related technologies continue to improve, including increases in accuracy and availability of orbits, the completion of additional Global Navigation Satellite Systems, and the construction of more robust GNSS receivers and antennas.

By closely following NGS 58 and 59 guidelines, a ten day static GPS survey campaign was conducted in Oregon during the summer of 2014. The resulting GPS baselines were processed in commercial software, Leica Geo

Office (LGO©), and the final static network was adjusted by least squares in MicroSurvey STAR*NET©. From the network adjustment, the estimated error in elevation at each station ranged from plus or minus 0.8 to 1.8 cm at 95% confidence, with an average error in elevation equal to plus or minus 1.3 cm at 95% confidence.

For comparison, the data was also post-processed in OPUS-Projects, a web-based program recently released by NGS for surveys involving multiple sites and multiple occupations. This comparison showed that a user-defined network in OPUS-Projects with a central hub tightly constrained to International GNSS Service (IGS) active stations yielded results most similar to those found in the commercial software. The average difference in elevations when comparing the commercial software solution with the final network in OPUS-Projects was only 5 mm.

Lastly, this paper makes recommendations on how NGS 58 and 59 could be optimized by changing network configurations and changing the 1.5 cm RMS and 2.0 cm Up screening requirement to a statistical outlier identification approach. Areas of future research are also identified such as incorporating GLONASS, analyzing solutions based on fewer observations, and including trivial vectors.

Height Modernization from Static GPS Networks in Oregon: Evaluating NGS Guidelines and OPUS-Projects

by
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I understand that my thesis will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my thesis to any reader upon request.

Darren J. Kerr, Author

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LIST OF ACROYNMS

ASCII	American Standard Code for Information Interchange
CORS	Continuously Operating Reference Station
FGDC	Federal Geographic Data Committee
GEOID12A	2012 Geoid Model, Version A
GIS	Geographic Information System
GLONASS	Global Orbiting Navigation Satellite System
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
GPSCOM	Helmert Blocking normal equation processor
h_{83}	Elipsoidal Height, NAD1983(2011) epoch 2010.00
H_{88}	Elevation, NAVD1988
IDB	NGS Integrated Database
IGS	International GNSS Service
ITRF	International Terrestrial Reference Frame
LGO	Leica Geo Office
MAG	NGS Benchmark CORVALLIS MAG STA-226
MST	Minimum Spanning Tree
N_{12A}	Geoid Height, GEOID12A
N_{new}	Project Geoid Model
NAD83	North American Datum of 1983

NAVD88	North American Vertical Datum of 1988
NGS	National Geodetic Survey
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
NSRS	National Spatial Reference System
NW	Northwest
OP	Online Positioning User Service Projects
OPUS	Online Positioning User Service
OPUS-DB	Online Positioning User Service Shared Database
OPUS-RS	Online Positioning User Service Rapid Static
OPUS-S	Online Positioning User Service Static
OR	Oregon
ORGN	Oregon Real-Time Network
OSU	Oregon State University
PAGES	Program for the Adjustment of GPS Ephemerides
PDOP	3D Position Dilution of Precision
PVC	Polyvinyl Chloride
RINEX	Receiver Independent Exchange Format
RMS	Root Mean Square
SD	Secure Digital
SINEX	Solution Independent Exchange Format
STA	Surveyed Station

TRI	Triangle Network Design
VDOP	Vertical Dilution of Precision
WGS84	World Geodetic System 1984

1 INTRODUCTION

Determining heights is a central topic in the field of surveying and geomatics. Given today's technology, traditional methods of determining heights may be replaced with more modern techniques that utilize Global Navigation Satellite System (GNSS) receivers, networks, and post-processing strategies. Due to the speed, accuracy, and ease of use, GNSS can result in significant cost savings over conventional surveying methods, such as geodetic leveling, for determining heights. Levels of accuracy, efficiency of data collection, and the skills needed to obtain data clearly make modern height determinations via GNSS a relevant topic for research and the surveying industry. The surveying community is acknowledging the growing benefits of obtaining orthometric heights via GNSS, and the National Geodetic Survey (NGS) has coined the phrase "Height Modernization" to describe this process.

As defined by NGS, "Height Modernization is an initiative focused on establishing accurate, reliable heights using Global Navigation Satellite System (GNSS) technology in conjunction with traditional leveling, gravity, and modern remote sensing information" (National Geodetic Survey, 2015b). The focus is on utilizing GNSS to derive heights in combination with more traditional methods. In fact, Height Modernization which includes the eventual implementation of a new North American Datum, is one of the key tasks assigned to NGS (Crawford, 2013). In order to fully implement this new North American Datum, improving the accuracy and resolution of the geoid throughout the United States is another key aspect of this new initiative. Height Modernization has already taken great strides in other countries, such as Canada, and it seems to be the future of elevation determination because the positives far surpass the perceived negatives (Brazeal, 2014).

The benefits of height modernization are developing accurate, reliable and most importantly up-to-date heights. Height modernization also allows for the verification of coordinates of old passive marks that may have moved, and it easily allows for surveyors to create new marks for control. Today, surveyors, emergency managers, scientists, the agriculture community, natural resource managers, engineers, and the developers of Geographic Information Systems (GIS) all require up-to-date heights (National Geodetic Survey, 2015b).

The concept of height must first be explained in order to fully understand Height Modernization. In order to discuss a position's height component, three surfaces of the earth must first be defined. The three surfaces of interest are the topographic surface, the ellipsoidal surface, and the geoid surface. The topographic surface is the ground surface on the earth and is easily visualized. The ellipsoidal surface is made up of an ellipsoid of revolution designed to best fit the earth's general oblate spheroid shape, and it is created by defining the earth's major and minor axes along with a flattening ratio. The ellipsoidal surface is the surface of a pure mathematical model and is smooth in its curvature compared to the other irregular surfaces. Lastly, the geoid is an undulating and equipotential surface that corresponds to "mean-sea level" over the oceans. Due to changes in topography and anomalies in the earth's gravitational field, the geoid is irregular and bumpy with peaks and valleys. If the earth had no differences in density or gravitational anomalies, then the geoid would be smooth much like the ellipsoid (Van Sickle, 2008).

Three heights that are critical to the field of surveying and define the relationships of these three surfaces to each other are orthometric height (H), ellipsoidal height (h), and the geoid height (N). Orthometric height (H) represents a height on the ground above or below the reference geoid surface, and is commonly called elevation. This height is the distance from a point on the topographic surface to the geoid following along a plumb line normal to the geoid (Zilkoski, 1990b). Because the path between these two points follows the plumb line, it is a slightly curved line due to the changes in the normal position from the geoid surface at different points (Zilkoski, 2010). The ellipsoidal height (h) is the distance from the topographic point to the referenced ellipsoid, and it is measured from the normal of the ellipsoidal surface up or down to the topographic point. Lastly, the geoid height (N) is the difference between the orthometric height and the ellipsoidal height for a particular topographic point. This height is also known as the geoidal undulation or the geoidal separation. In practice, the relationship is given by Equation 1:

$$H \approx h - N \qquad \text{(eq. 1)}$$

An example of this relationship is also depicted in Figure 1 below. The relationship in eq. 1 is only shown as an approximation due to several factors including the geometry of the associated height paths, distortions, modeling errors, and datum inconsistencies (Fotopoulos et al., 2003). However, the residuals of eq. 1 are usually submillimeter and are negligible in most surveying applications. When considering the entire earth, the values for geoid height range approximately from ±100m (Brazeal, 2014).

GNSS yields heights that are relative to the ellipsoid, whereas vertical datums are relative to the geoid. In the National Spatial Reference System (NSRS), the ellipsoid surface was realized by the latest national adjustment, NAD1983(2011) epoch 2010.00. In the NSRS, the geoid surface was realized by adjustment of geodetic leveling data, with the most recent vertical datum known as NAVD1988. NGS has also developed hybrid geoid height models for converting heights in NAD1983(2011) to NAVD1988. The most recent hybrid geoid model is known as GEOID12A. Actual data files for GEOID12A are available from URL:[http://www.ngs.noaa.gov/GEOID/GEOID12A/]. Equation 2 summarizes the relationship between the three aforementioned heights, with their current NSRS datums shown as subscripts.

$$H_{88} = h_{83} - N_{12A} \qquad \text{(eq. 2)}$$

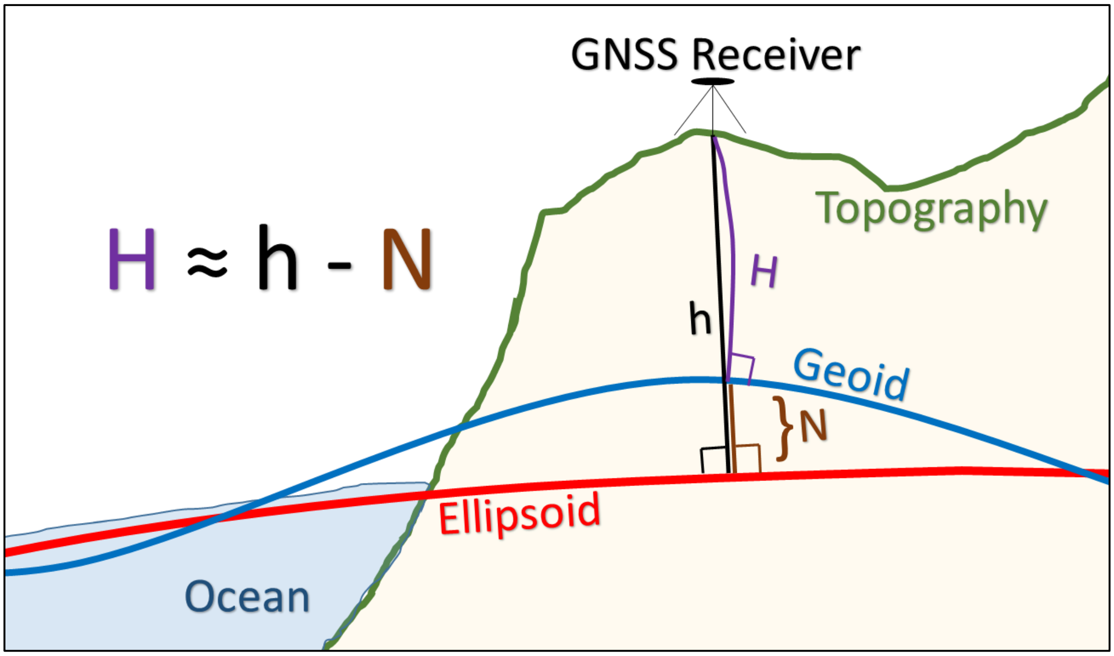

Figure 1: Elevation Adapted from Ellipsoid Height and Geoid Height

Deriving accurate heights, especially elevations, is of great importance to a number of scientific and engineering applications. Common uses for accurate elevations include: mapping flood hazards, determining sea level rise, developing nautical charts, precise terrain mapping, monitoring vertical crustal movements, studying subsidence and other ground deformation. One of the major factors that make determining and recording accurate elevations vital is knowing and predicting the effects of gravity at different elevations and the associated movement of water, soil, and other objects due to gravity's downwards pull toward lower elevations. Any scientific or engineering project that requires elevation data will require some degree of accuracy reporting that defines the uncertainties.

Presently, GNSS data is used to determine ellipsoidal heights, and geodetic leveling data is used for determining orthometric heights. As a vast simplification, GNSS surveying utilizes expensive and highly accurate antennas and receivers set up over known or unknown points. These receivers collect overhead satellite signals L1 (1575.42 MHz) and L2 (1227.60 MHz) and for some satellites L5 (1176.45 MHz). For accurate GPS positioning at least four satellites must be observed from a receiver, but more satellites help to provide redundancy and refine the position of the receiver. Positioning is determined by trilateration. This process is similar in concept from a sphere of signals outputted from the satellite and the point where three spheres intersect solve for a unique point in space. The fourth sphere from an additional satellite corrects for the receiver clock error and additional satellites help to reduce the area of uncertainty on the ground. Since the satellites are operating in the reference frame WGS84, the resolved X, Y, and Z position of a receiver can be determined from the Earth’s center of mass in WGS84. This X, Y, Z position can then be readily transformed into a geodetic latitude, longitude, and ellipsoid height by applying an iterative or closed form solution that accounts for differences from WGS84 and the desired reference ellipsoid system such as NAD83 (Crawford, 2013).

Geodetic leveling is the process of transferring height information via differential leveling from known reference points of elevation to unknown points. This is done by using leveling rods that are plumbed, and leveling instruments that read the rods either digitally or manually via magnifying optics. This process of differential leveling measures vertical differences of foresights and backsights in order to accurately determine the elevation of the leveling instrument at each setup, and it ultimately can calculate elevation differentials between the known and unknown points for a particular project. When conducted correctly and when blunders and systematic errors are removed, precise differential leveling can result in sub-millimeter precision depending on the length of the leveling line. Geodetic leveling remains a highly precise method for obtaining orthometric heights. However, geodetic leveling is a slow processes that requires highly trained surveyors who must carefully conduct and check their work. As a result, geodetic leveling is costly. Leveling is also prone to unique problems pertaining to changes in topography and moving great distances along lines of longitude. These problems can result in different elevation solutions due to changes in the deflection from the vertical and changes in gravity at different degrees of longitude. Another downside of geodetic leveling is that if an error occurs, the entire leveling line may need to be re-leveled.

With the development of geoid models, such as GEOID12A, GNSS can also be used to determine orthometric heights. Surveyors can obtain an orthometric height by first determining an ellipsoid height at a point, and then subtracting the geoid height from the geoid model at the point of observation. The accuracy of orthometric heights gained in this fashion are subject to all of the errors associated with finding the ellipsoidal height with GNSS, and errors in the model and in the interpolation of the geoid. Furthermore, errors in heights used for network control are also propagated to the newly derived orthometric heights via GNSS in a similar manner to conventional surveying (Satalich, 1996). With all of these error factors in mind, the accuracy of the geoid model is pivotal in ensuring reasonable values for orthometric heights, and the ability to derive elevations is primarily limited by a lack of accurate geoidal height information (Lee, 1993). For example, the current geoid model of GEOID12A has an average accuracy of 1.5 cm across the conterminous United States and directly effects the accuracy of any GNSS derived orthometric height (Crawford, 2013). There are a variety of techniques, including the use of well-planned networks, to reduce the errors involved in GNSS orthometric height determination. Generally, orthometric heights can be determined to centimeter level accuracy utilizing these techniques. Even though obtaining orthometric heights via GNSS is somewhat less precise than well executed geodetic leveling, GNSS surveying is significantly more efficient, easier, and less costly than geodetic leveling. In fact, cost savings of 40% or more may be achieved via GNSS height determinations compared to traditional techniques (Hajela, 1990).

NGS has even released specific and detailed recommendations and instructions on how to conduct a survey in order to determine both ellipsoidal and orthometric heights to ± 2 cm at 95% confidence using GPS. These guidelines were developed in partnership with federal, state, and local government agencies, academia, and independent surveyors (Zilkoski et al., 2008). NGS Technical Memorandum NOS 58 (hereinafter referred to as NGS 58) deals with ellipsoidal heights (Zikoski et al., 1997), and NGS Technical Memorandum NOS 59 (hereinafter referred to as NGS 59), published later in 2008, deals with orthometric heights (Zilkoski et al., 2008). By following these guidelines, errors in height measurements are expected to be no more than ±2 cm at 95% confidence. Zilkoski (2010) states that the danger of not following the guidelines reduces the ability to identify problems that may have occurred during a survey. In fact, David Zilkoski, the co-author of these guidelines and former Director of NGS states, “the purpose of NGS guidelines are to reduce, detect, and/or eliminate error sources” (Zilkoski, 2010).

1.1 Problem Statement and Research Objectives

A major objective of this thesis is to evaluate NGS 58 and 59 guidelines in light of modern advancements in GNSS technology, including GNSS hardware and post-processing software. Such research is necessary, as NGS 58 and 59 were published based on 1990s GPS technology and experiences. Since publication of these guidelines, GPS-related technologies continue to improve including: advancements in receiver accuracy, the completion of additional GNSS constellations such as Russia's GLONASS, the improved accuracy and availability of GNSS orbits, the development of real-time GNSS reference networks (RTN) covering much of the United States, and the construction of more robust GNSS receivers and antennas. In addition, NGS has developed and released significantly improved hybrid geoid models (e.g., GEOID12A), enabling higher accuracy computations of orthometric heights from GPS-measured ellipsoidal heights. Furthermore, NGS has released OPUS-Projects, a user-friendly cloud-based tool for performing baseline processing and least squares adjustments of simultaneous GPS occupations involving multiple sites. These advancements since the 1990's may allow for more efficient and possibly more accurate resolutions of GPS derived heights than the current guidelines of NGS 58 and 59.

Major objectives of this research are:

1. Evaluate the accuracy of NGS 58 and 59 using modern GPS receivers, today's satellite constellations, and current commercial baseline processing and least squares adjustment software
2. Provide recommendations for modification and/or possible optimization of the guidelines set forth in 58 and 59
3. Evaluate the accuracy of OPUS-Projects (OP) and determine if it is possible to derive heights using OP at the same accuracy level as heights derived following NGS 58 and 59 guidelines

4. Provide recommendations for future research

To accomplish these research objectives, a GPS height modernization survey was conducted in the Willamette Valley, Oregon, by carefully following guidelines recommended in NGS 58 and 59. Results of this survey are examined and findings are discussed. Afterwards, data collected during this survey were compiled into a variety of network geometries and adjusted in OPUS-Projects. Heights derived in OPUS-Projects are then compared with heights computed following NGS 58 and 59 guidelines.

1.2 THESIS FORMAT

This thesis has been formatted to follow Oregon State University's standard document format.

Chapter 2 provides a thorough review of key literature pertaining to technology, techniques, and applications of obtaining ellipsoidal and orthometric heights via GPS surveying techniques. This chapter also addresses other research completed in this area. This chapter also discusses errors involved in GPS surveying.

Chapter 3 presents the planning and execution of the static survey conducted in order to analyze the post-processing of both ellipsoidal and orthometric heights.

Chapter 4 focuses on post-processing of the survey data in order to determine heights while following NGS 58 & 59 guidelines. This chapter utilizes commercial post-processing software and an extensive statistical analysis of the data.

Chapter 5 centers on utilizing new NGS online software and different techniques of post-processing. It also experiments on different network designs.

Chapter 6 portrays the results of the different techniques. The 95% confidence statistics are also presented.

Chapter 7 discusses the key findings and makes recommendations for improvement and future research. Trends and an in depth analysis on the different survey methods are also presented. This chapter also offers recommendations for improvement of NGS 58 and 59.

Chapter 8 provides conclusions of this thesis.

Two appendices are enclosed that contain station and horizon photos (Appendix A) and visibility diagrams (Appendix B).

2 LITERATURE REVIEW

2.1 National Geodetic Survey's Guidelines for GPS Heights

Due to updated GPS technology, it is desirable to reevaluate NGS 58 and 59 guidelines every 5 to 10 years in order to determine if they need to be modified or if they can be optimized. Zilkoski states, “guidelines are modified as procedures, equipment, and models improve,” and that, “guidelines are much harder to get changed then to establish them” (Zilkoski, 2010).

2.1.1 Execution of NGS 58

In November 1997, NGS released *NOAA Technical Memorandum NOS NGS-58: Guidelines for Establishing GPS-Derived Ellipsoid Heights*. These guidelines provide recommendations for how to achieve ellipsoid height network accuracies of ±5 cm at 95 percent confidence, and ±2 cm at 95% confidence for local relative accuracies (Zilkoski, et al, 1997). Surveys executed following NGS 58 guidelines can be submitted to NGS for publication following a process known as “bluebooking.” NGS publishes the heights of such surveys to the nearest centimeter. Another benefit of following these guidelines is that the overall quality of the data can be assessed by showing repeatability, RMS, and loop closures (Henning, 2010). NGS 58 focuses heavily on the execution of a survey intended to achieve these results and has numerous requirements.

First, dual frequency GPS receivers are required for baselines greater than 10 km. This requirement along with all of the requirements in NGS 58 are designed to either detect or reduce error. For example, this specific guideline for dual frequency receivers is designed to reduce atmospheric errors. Additionally, the guidelines specify where possible, antennas used during a project should be identical. If a mixture of antennas are used then their phase center off-sets must be carefully corrected. Moreover, choke ring antennas that reduce the effects of multipath are highly recommended but not an explicit requirement (Zilkoski et al., 1997).

Second, NGS 58 specifies a survey hierarchy that consists of four levels, and the purpose of creating such a network hierarchy is to determine where problems in the data lie after post-processing (Zilkoski, 2010). The four levels include National Spatial Reference System (NSRS) Stations, Primary Base Stations, Secondary Base Stations, and Local network Stations.

NSRS Stations. These high accuracy stations must surround the project area in at least three different quadrants of the project, and must have NAD1983 coordinates in the most recent NSRS datum (i.e., NAD1983(2011) Epoch 2010.00). Allowable NSRS stations include active stations (i.e., CORS), and passive stations that were surveyed during regional HARN surveys. The NSRS marks must be spaced within 75 km of one another (see Figure 2) (Zilkoski et al., 1997).

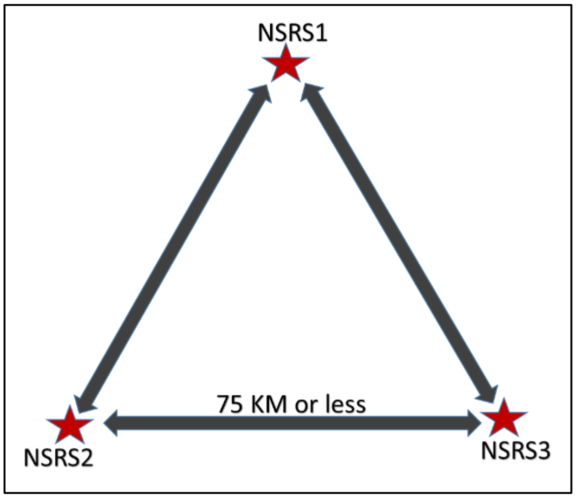

Figure 2: NSRS Spacing

Primary Base Stations. The next level is to densify the control network by connecting the NSRS marks to "primary" marks with baselines that cannot exceed 40 km (Figure 3). Each primary mark must also be connected to the nearest primary neighbor and nearest NSRS control mark within the specified distance requirements (Zilkoski et al., 1997).

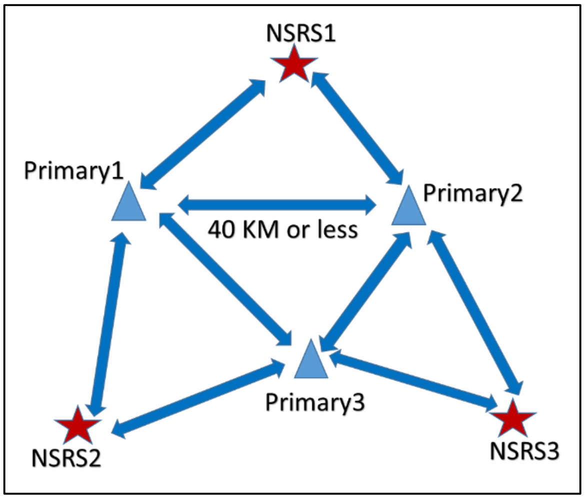

Figure 3: Primary Base Network and Station Spacing

Secondary Base Stations. The third level is to further densify the network by connecting the primary/NSRS marks with baselines to “secondary” marks at a spacing that shall not exceed 15 km (Figure 4). All secondary marks must also be connected to at least its two nearest primary or secondary neighbors. Each secondary mark must be linked back to two primary base stations along independent paths (Zilkoski et al., 1997).

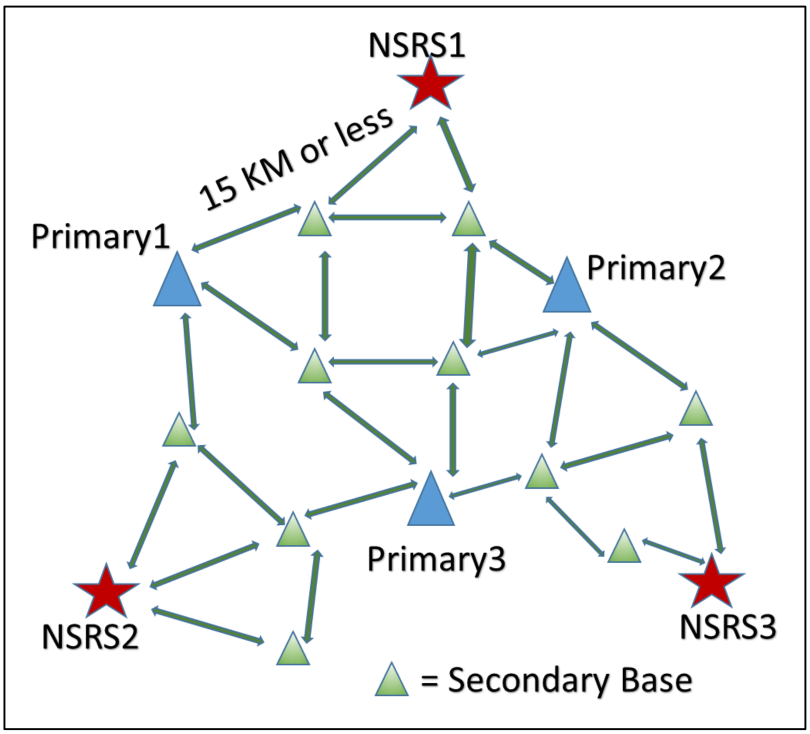

Figure 4: Secondary Base Network and Station Spacing

Local Network Stations. If needed, any other desired marks are connected to other marks with baselines that cannot exceed 10 km with an average spacing between local marks not to exceed 7 km (Figure 5). Each local mark must be linked back to two other marks (Zilkoski et al., 1997).

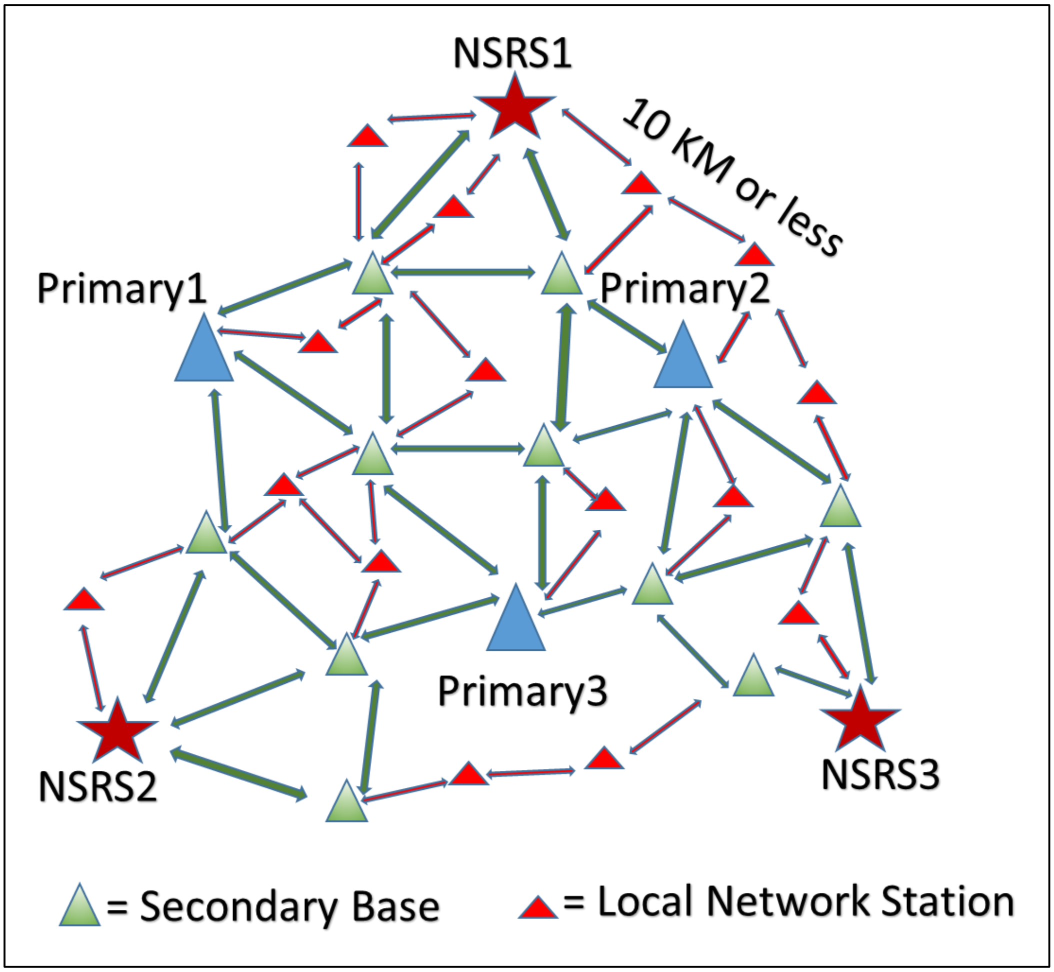

Figure 5: Total Network with NSRS, Primary, Secondary, and Local Stations

Observation times for each station is also specified. First, for baselines connecting the NSRS and primary base marks, GPS data shall be collected continuously and simultaneously for at least three, 5-hour sessions on three different days. For all other required baselines, NGS 58 requires that 30 minute sessions are conducted at least twice and on two different days. It is also recommended, but not required, that each session lasts for at least 45 minutes to ensure 30 minutes of good data was collected (Zilkoski, 2010). Moreover, all of these shorter sessions should be repeated at different times of the day, offset by 3 to 9 hours of time difference from the first observation session. This time offset ensures that the satellite geometry is significantly different for each repeat session and allows for different atmospheric conditions. Each of the baseline connections mentioned above must also be observed twice, so repeat baselines can be calculated and used to detect errors (Zilkoski et al., 1997). Redundant measurements from different satellite geometries helps to reduce multipath effects due to a specific satellite configuration (Henning, 2010). If the observations were not offset by time of day, studies show that very similar resulting coordinates can occur due to a similar satellite geometry yielding a high degree of precision. However, these same studies show that the resulting data may be inaccurate and unnoticeable as an error due to their precision (Zilkoski, 2010).

NGS also set requirements on numerous other collection criteria. For example, maximum Vertical Dilution of Precision (VDOP) should have a VDOP value less than 6 for at least 90 percent of each observation. VDOP is essentially an indication of the strength of overhead satellite geometry and its effect on trilateration vertical accuracy. Solving for VDOP results in factors ranging from 1 (ideal) to >20 depending on the number of satellites and their position overhead (Van Sickle, 2008). Also, epoch intervals must be taken at 15 second intervals or less, and the horizon mask angle should be recorded down to 10 degrees even though NGS requires post-processing to be set at 15 degrees. NGS also requires detailed meteorological data recordings for temperature, humidity, pressure, and general weather notes at the start, end, and sometimes at the midpoint of every observation. The purpose of collecting the weather data is to identify any major changes to weather fronts that could influence the data. Additionally, GPS antenna setup is crucial for success. Only fixed-height tripods are allowed, and the plumbing bubbles must be calibrated and checked at the beginning of the project (Zilkoski et al., 1997). Each antenna must also be oriented similarly across the network. Since the CORS are already oriented north, then each antenna should also be oriented north for the most consistent results (Meyer et al., 2006). Lastly, NGS requires either rubbings or detailed photos of each occupation of a station (Zilkoski et al., 1997).

2.1.2 Post-Processing According to NGS 58

NGS also implemented several requirements for post-processing. First, precise ephemerides, such as those provided by IGS, must be used for post-processing baselines. During baseline processing, the elevation mask should be set at a 15 degree angle to the horizon, and only baselines with fixed integers should be used during the final network processing. This requirement ensures that carrier phase based solutions are fixed and greatly increases accuracy over float solutions. Additionally, a model to account for tropospheric delays must be used (Zilkoski et al., 1997).

The network must be processed via a least squares adjustment and the quality of collected data shall be determined from adjusted baseline component residual plots and RMS values. Repeat baselines and loop misclosures should be checked and compared, and NGS requires that the RMS values for each processed baseline must not exceed 1.5 cm. Moreover, NGS requires that if the baseline ellipsoidal height difference exceeds 2.0 cm when comparing repeat observations, then the baselines must be re-observed (Zilkoski et al., 1997).

The actual NGS 58 memo was lacking some clear guidelines dealing with specifics on conducting least squares adjustments and the selection of constraints. This aspect of conducting adjustments, checking for outliers, and then re-adjusting until the final ellipsoid heights are determined will be discussed in greater detail during the post-processing section of this thesis.

Ultimately, if guidelines to determine ellipsoid heights at ±2 cm are not followed and bad baselines remain in the network, the errors will propagate when attempting to determine orthometric heights under the guidelines of NGS 59. Therefore, in order to determine accurate orthometric heights with GNSS, it is imperative to first determine accurate ellipsoidal heights (Zilkoski, 2010).

2.1.3 Background of NGS 59

In March of 2008, NGS released *NGS 59: Guidelines for establishing GPS-Derived Orthometric Heights*. This document was to be used in conjunction with NGS 58 in order to derive orthometric heights. In the preface of the memorandum, it states that NGS 59 is “loose” in tone because there are no strict standards. The reasons for this is that the document was not founded on an extensive scientific study, but it was based on experiences of several individuals within NGS and the surveying community. It also is not strict on standards because the “validation” of NAVD88 heights may be called into question due to the fact that NAVD88 was based on leveling to benchmarks or passive marks that are not tracked regularly. The earth is a dynamic surface due to tectonics, subsidence, uplift, and other factors, and changes in topographic positions over time can cause discrepancies with the elevations of passive marks. Lastly, benchmarks often disappear due to widespread construction which continues to create problems in following the techniques of NGS 59 (Zilkoski et al., 2008).

These guidelines are for campaign style surveys designed to either validate or create orthometric control on passive marks. The guidelines are also designed to almost always achieve the intended accuracy compared to the fact that similar accuracy can only occasionally be obtained via OPUS-Static solutions, which will be discussed in more detail below. Ultimately if the guidelines are followed, NGS claims that GPS is a viable alternative to classical geodetic leveling techniques for determining accurate orthometric heights (Zilkoski et al., 2008).

2.1.4 The 3-4-5 System of NGS 59

The process depicted in NGS 59 follows a 3-4-5 System that includes three basic rules, four control suggestions, and five procedures for estimating GPS-derived orthometric heights. The general workflow of the orthometric adjustment following this system is depicted in Figure 6. Each aspect of this system is also explained in greater detail in this section.

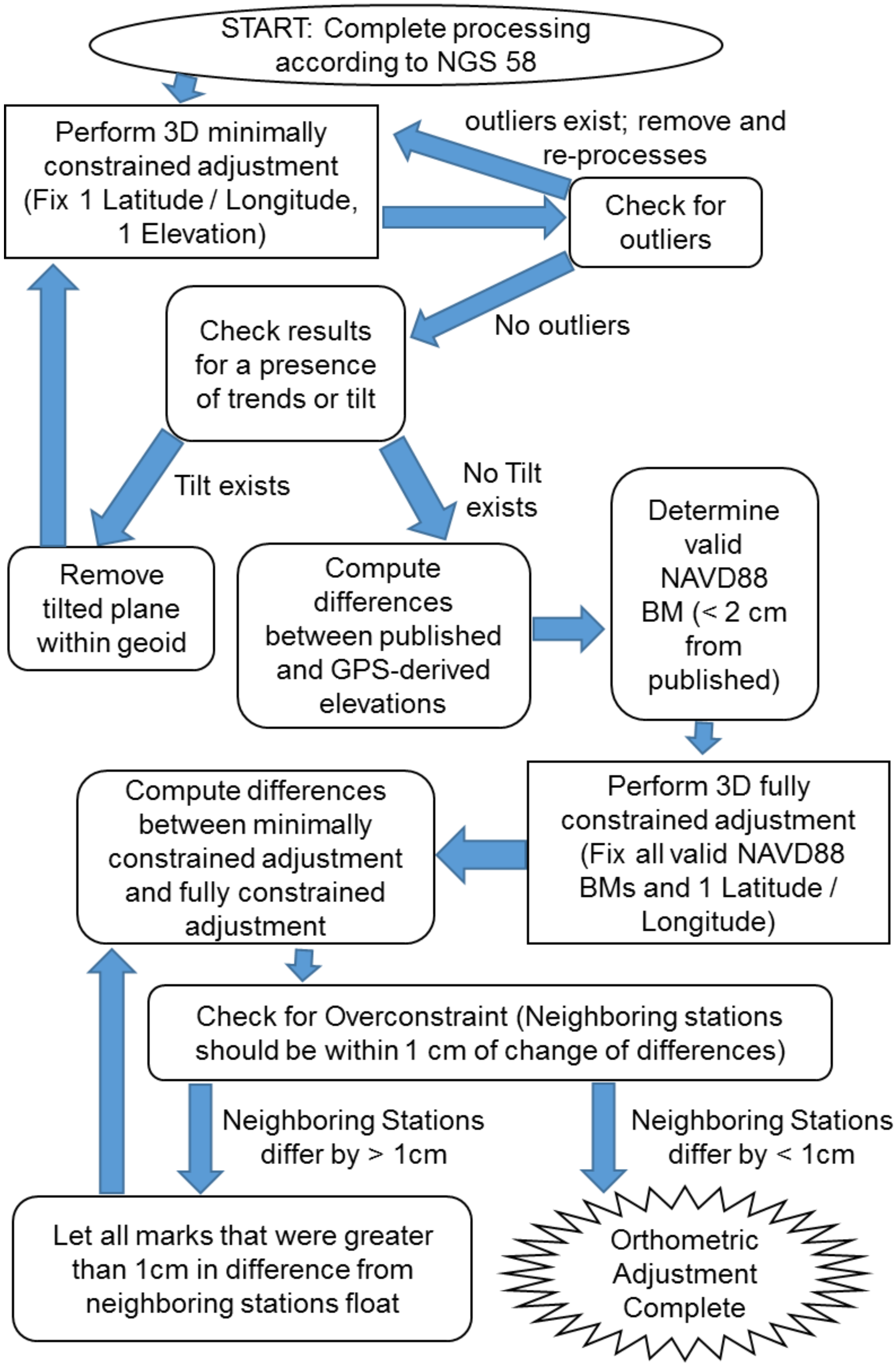

Figure 6: Orthometric Adjustment Workflow

The first aspect of the three basic rules are paramount. Rule one is that the network should follow guidelines of NGS 58 and all problems or outliers are eliminated from the network prior to starting any processing involving orthometric heights. Rule two is that the latest hybrid geoid model released by NGS, currently GEOID12A, must be used for computations, and the third rule is that the latest National Vertical Datum height values, currently NAVD88, should be used to control adjusted elevations (Zilkoski et al., 2008). The reasons for the three suggestions is to ensure that the most up to date models are used in order to ensure the highest degree of accuracy and conformance to current national reference frames.

The Four Basic Control Suggestions center on execution and/or planning principles that help to ensure more accurate final results. First, Suggestion One states that stations with valid orthometric heights (from the NAVD1988 datum) should be evenly distributed throughout the project. According to NGS, a valid orthometric height is considered one that has an adjusted, published Helmert orthometric height in the NGS database. Hereinafter, such a valid station will be referred to in this thesis as an “NAVD88 benchmark”. The next suggestion states that for projects less than 20 km on a side, the project should be surrounded with a minimum of four valid NAVD88 benchmarks, one in each corner. Similarly, the third suggestion is that for projects greater than 20 km on a side, distances between valid NAVD88 benchmarks must be less than 20 km apart. Lastly, Suggestion Four states that for projects located in mountainous regions, valid NAVD88 benchmarks near the lowest and the highest elevation points in the area should be occupied, regardless of the distance between them; furthermore, if possible, available NAVD88 benchmarks should be occupied throughout the range of elevation change (Zilkoski et al., 2008).

These Four Basic Suggestions for occupying NAVD88 benchmarks and their minimum spacing may be the most important planning factor for ensuring accurate orthometric heights via GPS. In fact after development of these guidelines, the primary limiting factor in determining orthometric heights with GPS is not the accuracy of the geoid; rather it is related to the presence or lack of valid NAVD88 benchmarks in a project area (Steinberg and Even-Tzur, 2008). Moreover, the distribution of NAVD88 benchmarks is important in verifying geoid height differences and should be planned throughout the network (Zilkoski, 1990a). A few years after NGS 59 was released, Zilkoski (2010) stated that at least one NAVD88 benchmark close to the center of the project, in addition to the other four suggestions, would further help to derive accurate orthometric heights in the project area. These suggestions are only meant to provide a minimum number of required benchmarks for the survey. Marks may be disturbed, and just as in conventional leveling, GPS networks should be tied to as many valid marks as possible for redundancy and increased accuracy (Milbert, 1991).

After implementing the Four Suggestions, the Five Basic Procedures of the 3-4-5 System focuses on post-processing techniques of the data. The first procedure is to perform a 3D minimally constrained least squares adjustment of the network focusing on orthometric heights and constraining the latitude and longitude of one NSRS control station and one NAVD88 benchmark. The next procedure is to check for outliers such as high residuals. NGS 59 doesn't define what an outlier is, and the methods used for this thesis will be discussed in detail later. In theory, the number of outliers should be low since most, if not all, of the poor baselines should have been removed when analyzing the network and deriving ellipsoid heights according to NGS 58. If outliers are found, procedures one and two should be repeated until all outliers are removed (Zilkoski et al., 2008).

The next two procedures focus on determining which published NAVD88 benchmarks are valid. Procedure Three is to compute the differences between the GPS-derived orthometric heights and the published orthometric heights on the NAVD88 benchmarks. Next, Procedure Four is to use the results from Procedure Three and determine which benchmarks are valid. Procedure Four is considered “the most important step” in NGS 59 (Zilkoski et al., 2008). In order to be considered valid, a benchmark's GPS-derived elevation must agree with the published elevation by 2 cm. After Procedure Four, the data should be checked for any patterns or trends with the differences from published to GPS-derived heights. Analysts should take caution if a spatial pattern is found when evaluating differences in published heights with GPS-derived heights. If a pattern in height differences travels in one direction, then a tilt in the geoid may be present and would need to be removed for the most accurate results (Zilkoski, 2010).

Procedure Five is to perform a final, fully constrained adjustment by fixing the latitude and longitude of one NSRS control station and the orthometric heights of all of the valid NAVD88 benchmarks determined in Procedure Four. After completing Procedure Five, the results must be checked for over constraint. To check for an overly constrained network, the differences between the results from the fully constrained network should be compared to the minimally constrained network. After calculating the differences between these two adjustments, neighboring stations should not be greater than 1 cm in difference, and a difference in 2 cm is a clear sign that invalid or incorrect vertical control was fixed (Zilkoski et al., 2008). Zilkoski later stated that analyzing the network for unnecessary constraints is not an exact science, and one should look for differences that emerge when subtracting minimally from fully constrained networks (Zilkoski, 2010).

After conducting all of the steps in NGS 58 and 59, the final coordinates for each mark are determined. For each mark, the latitude, longitude, and ellipsoidal height will be taken from the final results following NGS 58, and the orthometric height will be taken from the NGS 59 results. Any ellipsoidal heights derived from the NGS 59 process are not maintained, although they can be used to develop a local geoid model for the project area. (Zilkoski, 2010). The final NGS 59 network adjustment moves the three dimensional GPS network up or down to coincide with the local vertical datum. Ellipsoidal heights derived from the NGS 59 process also include any height bias from the vertical datum and the geoid model. Since the purpose of the final NGS 59 adjustment is to derive the orthometric heights, this bias is not a problem (Milbert, 1991).

Large differences between GPS-derived orthometric heights and the published leveling derived orthometric heights may not be due to errors in the GPS network. Instead, large errors could be due to benchmark movement, misidentified stations, inconsistent vertical datums, incorrect published control, or poor geoid models. One way to check movement of benchmarks is to perform a new set of precise leveling between two or more bench marks and compare those results to published values (Zilkoski, 1990b). The earth is forever in a state of movement and old published elevations lose reliability over time.

After reviewing the procedures and only a few years after the release of NGS 59, Zilkoski (2010) admitted some of the NGS 58 and 59 requirements may need to be modified. First, Zilkoski suggested that the guidelines may need to be modified to account for the continuing emergence of active stations and CORS. Spacing requirements to CORS near a project area needs further research. He also acknowledged that OPUS Projects needs to be evaluated in relationship to NGS 58, but the quality of data could be directly related to the availability of CORS in any given area. Lastly, Zilkoski re-iterated that if the guidelines are ultimately changed, baselines still ought to be measured at least twice. He felt that repeat observations must still occur on a different day at a different time of day. This repeat observation requirement is needed to ensure that enough data is collected from different satellite geometries in order to more fully model random errors in GPS observations (Zilkoski, 2010).

2.2 Projects Following NGS 58/59

Several projects that occurred either before the release of NGS 58/59 or shortly after release followed most of the procedures outlined by NGS. First, an extensive study conducted by NGS and Caltrans occurred in San Diego in 1993 that heavily influenced the final guidelines published by NGS. Actually, NGS 59 even references this project on page 7 of the NGS 59 memo. This project recommended a total of 10 specific steps in order to derive orthometric heights via GPS. Most of these steps are similar to the final process within both NGS 58 and 59 with a couple additional steps. For instance, this project focuses on analyzing the project area in greater detail during the planning phase. It states that detailed analysis of both the geoid and the leveling data should be conducted to ensure that proper control stations are utilized (Zilkoski, 1993). Improvements in the geoid model help to reduce this step of analyzing the geoid prior to conducting the procedures in NGS 58 and 59, but analyzing the level lines may help to avoid accidently using orthometric control that was not valid in the final adjustments. However, errors in orthometric control may not be detected until after the entire network is processed, and attempts to identify bad level data prior may not filter all possible bad marks.

This San Diego project also resulted in several conclusions that were later incorporated into NGS 58 and 59. For instance, the project stated that the overall accuracy of GPS orthometric heights relies on the accuracies of GPS-derived ellipsoidal heights, the geoid model, and the leveling-derived NAVD88 benchmarks. Additionally, the project conclusion stresses that results should be evaluated using loop misclosures and repeat base line differences. Lastly, the project highlights the need to occupy all stations at least twice in order to detect, reduce, or minimize errors (Zilkoski, 1993).

Another project that influenced the guidelines of NGS 58 and 59 occurred in 1995 in Maryland. This project consisted of 18 monuments and resulted in a solution accurate to ± 2 cm. This project showed that baselines less than 10 km ensured that the 2 cm standard could be reached. The results of this project also show that the standards can be achieved due to the greater availability of satellites, more accurate satellite orbits, dual frequency carrier phase data, improved antennas, and improved data processing techniques (Henning et al., 1998).

Another project greatly related to NGS 58 and 59 was a survey project conducted in 1996 in Vermont (Martin, 1998). The goal of this survey was to determine a first order horizontal control network. However, while executing procedures required for horizontal accuracy, the investigators also found a high degree of vertical accuracy in their survey. Overall, the project was able to derive elevations to ±2 cm of the elevations published in the NGS database. The project found that several of the marks utilized were also used in the derivation of GEOID96 (i.e., the hybrid geoid model in use at the time). This resulted in a degree of accuracy that may not be achievable in areas where few marks were used when developing the current geoid model (Martin, 1998). This project also didn't follow any strict repeat baseline standards such as the 2.0 cm requirement set in NGS 58. Instead, Martin (1998) used a statistical analysis method in order to identify repeat baselines as outliers. During this project, the investigators were not able to collect a repeat baseline on every adjacent station. Nonetheless, the results suggest that if the network design is robust enough, then enforcement of the repeat baseline requirement is not necessary. Moreover, vertical loop closure analysis and a traditional network design can be used to evaluate the data instead of requiring a precise, repeat baseline for all baselines in a project. This would clearly reduce the amount of field observations and cost (Martin, 1998).

2.3 Other Research on the Accuracy of GPS-Derived Heights

Other attempts to determine orthometric heights via GPS observation have occurred with mixed results. It appears that the intended accuracy of 2 cm can be reached while following the stringent requirements of NGS 58/59, but can this level of accuracy be accomplished with less effort? Other research in the area has not conclusively answered this question, because a majority of other projects paid little attention to the requirements of NGS 58/59 or the reasoning behind the requirements. However, some recommendations or principles from these other projects are similar to the reasoning for NGS guidelines.

One project that attempted to derive orthometric heights via GPS occurred in Finland in 1997. The project consisted of two different survey areas over the course of multiple days and the residuals in the first area were ± 1.7 cm and 1.4 cm respectively while using a local geoid model. However, when using a global geoid model the RMS values varied from ± 4.0 cm to 8.0 cm (Ollikainen, 1997). Therefore, this project's findings support the claim that a local geoid is better than models at a larger scale, because it more accurately reflects the actual geoid at any given point. This project re-iterated the fact that precise levelling cannot be solely replaced by GPS, but the findings did make recommendations on when GPS could be used for determining orthometric heights. Specifically, the project concluded that the accuracy of GPS heights over moderate distances was approximately ± 9 mm + (0.3 mm ÷ distance of baseline in km). The final recommendations were that lower order leveling could be replaced by GPS on lines longer than 50 km (Ollikainen, 1997).

Another study that was conducted in Southern California in 1996 found that the largest differences in their network was greater than 10 cm and was associated with control stations at higher elevations or at marks that were in close proximity to mountains (Satalich, 1996). The largest difference between control stations was about 1000 meters in height. These findings coincide with the NGS recommendation to use more benchmarks when surveying in mountainous terrain. For the survey conducted in this thesis, the difference from the lowest mark to the highest was just under 544 meters, and the problems mentioned in this study from 1996 were relatively avoided. The Satalich (1996) study also focused on demonstrating the repeatability of GPS observations, and it recommended performing repeat observations and independent checks such as differential leveling for some marks in a project. Repeat observations are also required by NGS for their usefulness in identifying problems with any one sessions that could impact the overall solution.

A recent study in 2013 within the Czech Republic consisted of 19 points with 5 control points and covered a 20 x 60 km area. The study resulted in a time threshold recommendation for static observations. In this study, it was concluded that a 4 hour long session allowed consistent height repeatability (Baryla et al., 2013). Although 4 hour sessions will most likely yield excellent results, the project conducted for this thesis shows that repeatable results can be obtained from much shorter sessions.

A different study was conducted in San Diego in 1995 attempting to derive orthometric heights from GPS. An interesting finding in this study was that GPS baseline processing software generated over optimistic estimates of precision. This study also concluded that significantly smaller RMS values can be expected from surveys with shorter GPS vectors (Milbert and Parks, 1995).

NGS 58/59 also addresses baseline length concerns in their requirements, and these were followed for the survey's execution.

A similar project to deriving orthometric heights from GPS was conducted in Illinois in 1996 and focused on modeling local geoids. The findings of this project recommended a principle that is also recommend by NGS. In specific, the project recommended that fixed control points should be distributed evenly throughout the project's area in order to reduce any irregularity of the geoid (Fidis et al., 1996). Similarly, the spacing between constrained orthometric marks was taken into consideration when selecting constraints for the final orthometric processing for this thesis.

2.4 Error Sources for GPS Surveying

Prior to expanding on the specifics of the static survey execution, a thorough explanation of error sources for GPS surveying will be presented. There are numerous errors involved with GPS surveying, and research has still yet to identify all possible sources of error. However, the main errors involved will be discussed below. Three types of errors include blunders, systematic errors, and random errors. Random error will be addressed later and will be accounted for and defined by performing least squares adjustments during post-processing. Some common blunders will briefly be discussed below and the process for attempting to identify unknown blunders that occurred during the survey will be addressed later.

GPS errors include several errors that must be resolved in order to obtain a mathematical position on the earth. Since GPS receivers can resolve position according to the frequencies of the satellite signals, an equation of errors that must be resolved in order to obtain a position are presented in the carrier phase observable equation and is depicted in Equation 3 (Van Sickle, 2008).

$$\Phi = \rho + d_\rho + c(dt - dT) + \lambda N - d_{ion} + d_{trop} + \varepsilon_{m\Phi} + \varepsilon_\Phi \qquad \text{(eq. 3)}$$

where

Φ = carrier phase measurement

ρ = true range

d_ρ = satellite orbital errors

c = speed of light

dt = satellite clock offset from GPS time

dT = receiver clock offset from GPS time

λ = carrier wavelength

N = integer ambiguity in cycles

d_{ion} = ionospheric delay

d_{trop} = tropospheric delay

$\varepsilon_{m\Phi}$ = multipath

ε_Φ = receiver noise

If these errors are left uncorrected, they will introduce a large magnitude of error to the final GPS derived position. An example of how much error these factors can introduce is presented in Table 1 (Crawford, 2013).

Table 1: Summary of Carrier Path GPS Error Sources

Error Source	**Resulting Magnitude (m)**
Satellite Clocks	2
Orbit Errors	2.1
Ionosphere	5
Troposphere	0.5 (model)
Receiver Noise	0.3
Multipath	1
Phase Center Variation	0.1

Satellite Clock errors are mainly mitigated via relative positioning of differential GPS, and orbit errors are mainly mitigated via utilization of accurate ephemerides. Orbit errors present additional errors than just the orbits involved. For instance, the constellation of the satellites presents several error sources including errors with the satellite orbits, health of the satellite vehicles, number of overhead satellites available, and geometric distribution of the satellites (Zilkoski, D and L. Hothem, 1989). Satellite geometry is one of the reasons that the error budget is higher for vertical solutions versus horizontal solutions, and is due to receiving satellite signals from only above and not below the horizon. As a result, horizontal coordinates can be resolved from vectors of several directions in their respective plane, but vertical coordinates can only utilize vectors in half of the vertical plane from the overhead satellites.

Several techniques help to reduce errors resulting from the overhead satellite constellation. Pre-planning GPS observations to ensure the best satellite geometry or lowest PDOP is essential. Additionally, taking repeat GPS observations of the same baselines at different times of day ensures a completely different satellite geometry and helps to more fully account for these errors. Errors in the satellite orbits can be reduced by using precise ephemerides during baseline processing. Precise satellite ephemerides are released by the IGS at the following URL: [ftp://cddis.gsfc.nasa.gov/].

Another error source relating to the constellation of satellites is due to reduced visibility because of overhead obstructions (Zilkoski and Hothem, 1989). Overhead obstructions should be considered in any survey to ensure that an appropriate level of PDOP is obtained if some satellite signals are lost due to the physical blockage. In addition to poor PDOP concerns, large overhead obstructions may cause significant multipathing errors. Some research has also shown that overhead power lines can cause noise up to an additional 2 cm. However, this noise can be greatly reduced by simply collecting GNSS data for longer-duration sessions on marks beneath power lines (Crawford, 2013).

Another major source of error in GPS surveying is atmospheric delay of signals caused by the ionosphere and the troposphere (Crawford, 2013). Errors resulting from refraction of the satellite signals moving through the troposphere is one of the most limiting factors of geodetic applications of GPS (Ollikainen, 1997). The best way to mitigate this error source is by mathematical modeling of the atmosphere, and by double-differencing of short baselines. Of course, modeling is not foolproof. For example, an error of 1 centimeter in modeling the tropospheric zenith delay can result in a vertical position error of about 3 cm (Brunner, F and W. Welsch, 1993). Research has shown that by applying a masking angle of 15 degrees during baseline processing, the Saastamoinen tropospheric delay model results in errors due to tropospheric delays of 5 mm or less (Brunner F, and W. Welsch, 1993). This model was selected, as discussed in the baseline processing portion of this thesis.

Changes in the atmosphere over time should also be considered. In fact, research has shown that the repeatability of derived coordinates over the course of 4 seasonal campaigns varied from 1.2 cm to 2.4 cm vertically (Dodson, et al., 1996). Therefore, consideration to the length in days of a survey is a consideration to ensure that the season is not significantly different from start to finish. It is also important to identify the passage of weather fronts moving through the survey area, because weather fronts may cause signal delay to vary by greater than 3 cm over a one hour period, resulting in ellipsoidal height error exceeding 9 cm (Marshall, et al., 2001). The ability to identify fronts is why NGS 58/59 requires that field measurements of temperature, barometric pressure, humidity, and general weather observations are taken before, after, and sometimes in the middle of long sessions. These field measurements should only be used for weather front identification purposes, because research has shown that utilizing observed meteorological values in calculations instead of tropospheric models results in greater error (Brunner, F and W. Welsch, 1993). Moreover, significant change in regards to temperature, pressure, or humidity might not be detected by busy field crews unless those measurables are recorded (Meyer, et al. 2006). Regrettably, if a weather front is detected to move through a survey area, the best way to ensure accurate data is to re-conduct the survey at a different time.

Other errors that affect the overall accuracy of orthometric heights via GPS include: errors in regards to the lack of available vertical control and error in geoid heights or the model used to replicate the geoid (Lee, 1993). Other errors that may be introduced are related with: session length, baseline length, elevation mask, and accurate antenna height measurements (Satalich, 1996), differences between processing software packages (Zilkoski and Hothem, 1989), and even unique events, such as birds landing on a receiver (Crawford, 2013).

The accuracy of the heights held for vertical control also present errors. Due to the high cost of geodetic leveling, a significant number of passive marks in the United States have not been re-leveled for decades. Modeled vertical deformation of these passive marks may also be inaccurate, because models are based on active stations while the true deformation on passive sites is not measured. (Brazeal, 2014). The local availability or lack thereof of marks also creates a problem. In some areas marks are destroyed from urbanization and accidents, and in other areas no marks exist for vertical control. Great effort is required to ensure adequate spacing of vertical control for a GNSS height modernization survey. Also, checking the validity of the control coordinates during post-processing remains an essential step for accuracy.

Errors within the geoid present a problem for GPS surveys, particularly when attempting to derive orthometric heights. Therefore, it is critical that the most accurate geoid model is used, and a locally derived geoid model may be more accurate than a global or a national model. However, this study utilized the most up-to-date national geoid model, GEOID12A. GEOID12A is the proper hybrid geoid model for converting ellipsoid heights in NAD1983(2011) Epoch 2010.00 to orthometric heights in NAVD1988.

Common blunders that could occur include: misidentifying a benchmark, poor height measurements of an antenna set above a mark, and failure to level the antenna over a mark. The key to avoiding blunders is proper training and careful work. Surveyors should always strive to check their work during execution and search for possible blunders by conducting an outlier analysis which will be discussed later in more detail. In order to eliminate the error in poor height measurements of an antenna, fixed 2 meter tall tripods with calibrated levels were used for the survey in this thesis.

Surveyors conducting static GNSS work should even be wary of birds. Research has shown that birds landing on an antenna can result in vertical errors at the centimeter level (small birds). For large birds, decimeter level vertical errors may occur (Crawford, 2013). During one session of this study, a large bird was discovered perched on one of the antennas.

Heights obtained from GNSS networks are 1.5 to 2 times less accurate than horizontal coordinates due to weaknesses in satellite geometry and atmospheric delays caused by the ionosphere and the troposphere (Crawford, 2014). This study will show that by careful GNSS survey work, positioning errors can be kept to approximately 5 mm to 1 cm horizontally, and 2 cm vertically at the 95% confidence level.

3 GPS STATIC SURVEY EXECUTION

3.1 NETWORK DESIGN FACTORS AND PRE-SURVEY PLANNING

Planning prior to executing a multi-session and multi-day survey is key to success and perhaps one of the most important steps. For this study, the first step was to plan a survey to meet the criteria for spacing for NSRS, Primary, Secondary, and Local stations according to NGS 58 and 59. An additional end result of this project was to utilize one mark located on the campus of Oregon State University which is often occupied for teaching undergraduate and graduate level surveying classes. This mark is used frequently because it has a published elevation in the NGS database, but it does not have a previously bluebooked northing, easting, and ellipsoid height. This required mark is designated CORVALLIS MAG STA-226 in the NGS integrated database (IDB) and will be called MAG in all further discussion within this thesis. Since this mark is heavily utilized on campus, accurate northing and easting coordinates along with its ellipsoid height was a requirement for this study.

Other than following NGS 58/59 and observing MAG, implementing a new control mark with a monument on a high point within the Oregon State University managed McDonald-Dunn Forest was also an objective. The specifics of setting this monument will be discussed later, but for this requirement, a new mark named PEAK was set and observed during the campaign.

With the end goal of observing both MAG and PEAK at the local level, the first step in mark selection was identifying NSRS stations to act as the outer control for the survey. According to NGS 58, the selected NSRS stations had to be within 75 km of each other. Following NGS 58 guidelines, nearby passive marks that were observed during regional HARN surveys were chosen that also had minimal overhead obstructions and easy access for surveying. The number of eligible benchmarks fitting this criteria was rather low, but three stations were eventually selected. Specifically, the NSRS stations chosen for this survey (Table 2) were D728 near Halsey, U727 near Blodgett, and NESMITH located near the Polk County Fairgrounds in Rickerall, OR.

Next, a list of benchmarks with published orthometric heights was obtained to ensure that several marks with NAVD88 heights were ultimately chosen to satisfy the spacing requirements for the final survey network. One of the challenges in planning the survey was to find an adequate number of marks with published orthometric heights. Unfortunately, there was a sparse distribution of such marks in the study area that had minimal overhead obstructions, and a large number of benchmarks in the IDB were not found due to construction or vegetative overgrowth. One of the greatest challenges was finding benchmarks in the northern end of the project area that met the 20 km spacing suggestion and had minimal overhead obstructions. One station, J99, located on the northern end of the project and somewhat south of Monmouth, OR, was used despite the fact that it was located under a moderate tree canopy. In the end, thirteen of the twenty total marks selected had published NAVD88 elevations determined by first or second order leveling.

The NGS Data Explorer website was used extensively to find passive marks with published NAVD88 elevations. The site can be accessed at the following URL: [http://www.ngs.noaa.gov/NGSDataExplorer/]. The site plots the approximate location of benchmarks in the NGS IDB on top of Google Maps imagery. The tool also has hyperlinks to each mark's datasheet and proved highly useful in the planning phase of identifying potential marks for observation.

The next task was to select the primary network marks that would be linked to the NSRS and each other via the 3-day, 5-hour static observation sessions. The preference was to select marks in the NGS IDB with a published NAVD88 elevation, excellent overhead visibility, and high accessibility. The NGS 58 spacing requirement of 40 km between a primary station and its nearest NSRS station or primary station neighbor was also followed. Three marks were selected as primary base stations (Table 2). These marks are named N99-Reset near Airlie, S714 near Albany, and BICKFORD located at the Corvallis Airport.

Lastly, the secondary and local stations were selected. The spacing requirements for these marks is 15 km for secondary to primary or secondary to secondary stations. For the final network, the NGS 58 requirement is that baselines to local marks must be under 10 km long, with an overall average length under 7 km. In the end, the planned network had an average baseline length of 6.56 km to local stations, with the largest local to secondary station distance equal to 9.89 km. The same criteria used for selecting primary marks was used when selecting secondary marks. The final selected secondary stations used during the survey consisted of J99 near Monmouth, Y683 in Adair Village, BEEF, located on the OSU cattle ranch, PRICE located east of Hoskins, Z714 located south of Wren, CORVA, located in Corvallis, and D728 located near Shedd. The final selected local stations (Table 2) consisted of PEAV located in the OSU Peavy Arboretum, T714, in Corvallis, G287 in the Kings Valley, Q388-RESET located south of Corvallis, J54 located in Philomath, PEAK set near McCulloch Peak in the McDonald-Dunn Forest, and CORVALLIS MAG STA-226 located on OSU campus.

All potential marks for observation were visited prior to final selection, and photographs of the overhead conditions were taken. Some marks identified early on as possible candidates were not found and taken off of the selection list. Some marks were even tested for possible multipath prior to the full network survey by collecting and evaluating data from 2 hour static observations. The complete list of final chosen marks is listed below in Table 2, and a map depicting there general locations is presented in Figure 7.

Table 2: Marks Selected for Case Study

STATION	NGS IDB PID	NETWORK CATEGORY	PUBLISHED NORTHING (m) #	PUBLISHED EASTING (m) #	PUBLISHED ELLIPSOID HEIGHT (m)*	PUBLISHED ELEVATION (m)^	FBN / HARN / CBN	VERTICAL ORDER / CLASS	USED TO MODEL GEOID12A	STABILITY
U727	QE1564	NSRS	107,599.766	2,258,120.919	204.742	226.783	YES	1.2	YES	B
G728	QE1488	NSRS	85,650.514	2,292,155.683	61.309	84.421	YES	1.2	YES	B
NESMITH	QE2664	NSRS	143,466.297	2,284,683.294	41.519	63.923	YES	3	YES	C
BICKFORD	QE0656	PRIMARY	95,243.443	2,278,864.617	52.329	75.072	NO	1.2	YES	C
S714	QE1579	PRIMARY	111,599.871	111,599.871	43.603	66.464	NO	1.2	YES	B
N99-RESET	QE2734	PRIMARY	125,462.708	2,272,519.976	53.344	75.430	NO	3	YES	C
J99	QE0722	SECONDARY	129,935.724	2,284,668.071	53.898	76.442	NO	2.0	YES	C
Y683	AI2011	SECONDARY	117,245.751	2,284,523.133	71.990	94.60	YES	3?	YES	C
BEEF	AJ1539	SECONDARY	116,401.679	2,279,988.252	120.491	142.9	NO	NONE	NO	C
PRICE	AJ1549	SECONDARY	116,815.522	2,268,774.043	113.602	135.7	NO	NONE	NO	C
Z714	QE1571	SECONDARY	104,380	2,268,370	UNKNOWN	171.126	NO	1.2	NO	B
D728	QE1485	SECONDARY	92,359.0	2,292,487.8	UNKNOWN	79.193	NO	1.2	NO	B
CORVA	AI6289	SECONDARY	106,082.946	2,277,508.135	103.471	126.017	NO	1.2	YES	B
G287	QE0742	LOCAL	109,659.140	2,267,795.198	179.939	202.076	NO	2.0	YES	C
J54	QE0621	LOCAL	101,609.349	2,275,282.608	63.701	86.285	NO	1.2	YES	C
Q388-RESET	RESET	LOCAL	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	NO	NONE	NO	C
MAG	QE0636	LOCAL	103,730.300	2,279,875.800	UNKNOWN	70.096	NO	1.2	NO	C
T714	QE1576	LOCAL	105,023.833	2,282,052.127	46.752	69.481	NO	1.2	YES	B
PEAV	N/A	LOCAL	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	NO	NONE	NO	D
PEAK	N/A	LOCAL	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	NO	NONE	NO	D

Northing/Easting: NAD1983(2011) Epoch 2010.00 SPC, Oregon North Zone, meters
* Ellipsoidal height: NAD1983(2011) Epoch 2010.00
^ Elevation: NAVD1988

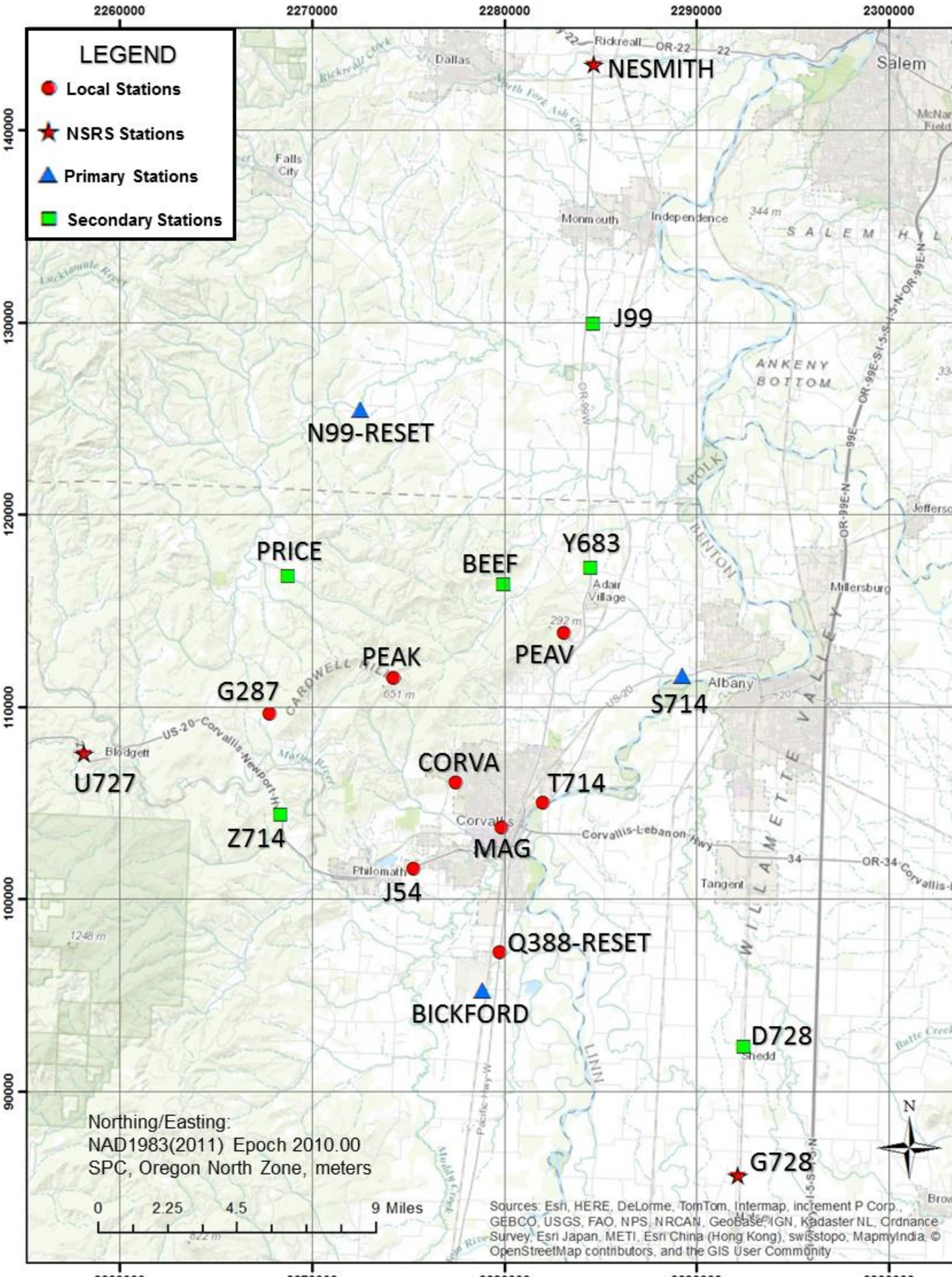

Figure 7: Project Area Map

3.2 EQUIPMENT

For this survey, five dual frequency Leica GS-14 integrated GPS+GLONASS receiver/antennas were used. External 12V batteries were connected to each receiver to provide adequate power for the long-duration observations. For every occupation, GPS+GLONASS RINEX format data was collected and stored on a removable SD card inserted in each antenna. A Trimble R8-Model 2 integrated GPS+GLONASS receiver/antenna was used to collect data on only one station, NESMITH in order to cover all NSRS and Primary Stations simultaneously. Otherwise, only the Leica GS-14 receivers were used to collect GPS+GLONASS data on the other stations.

Two meter Seco fixed-height tripods were used throughout the survey in order to comply with NGS requirements. Prior to the survey, the fixed-height tripods were calibrated and the bubbles were checked. Additionally, in order to increase stability to the setups, 30 lb. heavy duty sandbags designed to stabilize pop up tent canopies were set on each leg of the fixed-height tripods.

Other equipment utilized included three different types of digital psychrometers to record weather data namely a Kestrel 4500 BT, several Omega RH83, and a VWR Traceable ISO 17025. Additionally, a Suunto Tandem compass and clinometer was used for measuring overhead obstructions and sketching visibility diagrams. Lastly, a unique adapter (Figure 8) was assembled from brass sprinkler connectors and placed on B stability monuments. This adapter ensured faster and more stable setup of the 2 meter tripods on all B stability monuments; the adapter was created based on a modification of an adapter developed by NW NGS advisor Mark Armstrong. The adapter was placed on top of the benchmark's rod, and the opening on the adapter head was big enough to allow for the point of the 2 meter tripod to rest on top of the mark without slipping (Figure 8).

Figure 8: B-Stability Monument Tripod Tip Adapter


Figure 9: Tripod Tip Adapter Setup

All equipment was field tested on the campus of OSU, and GPS data was collected and post-processed as a rehearsal prior to conducting the survey in order to increase efficiency in the field. The testing ensured that the data was successfully logged and transferred into software packages for baseline processing. Most of this testing occurred concurrently with the planning phase of the network. An example of a complete station setup is depicted in Figure 10 below.

Figure 10: Example Setup at Station MAG

3.3 MONUMENTS

Most of the marks observed in this survey are published in the NGS IDB and are classified as B or C stability monuments. NGS defines monuments according to their stability on a scale from A-D. An A monument is usually encased in bedrock and expected to most likely maintain position. On the opposite end of the spectrum, a D monument is set via an unknown or nontraditional NGS procedure that may move in position with time (Henning, 2010). NGS has guidelines on how to set monuments according to the different classes of stability. For example, B Stability (Figure 11) benchmarks consist of stainless steel rods driven to refusal and encased in a PVC pipe layered with grease, within a larger PVC piper layered with sand. This pipe column is placed a total of 1.1 meters into the ground, and is also set in sand near the bottom and encased in concrete. This construction helps to ensure that ground freeze and thaw cycles do not shift the position of the monument. Lastly, the mark is encased with a logo cap that flips up for access to the rod. The exact observable point is a divot on top of the stainless steel rod (Smith, 2010).

Stability C monuments (Figure 12) are essentially brass disks set in concrete and may lack the overall stability of A or B monuments over a long period of time. Photos of every mark used in this survey are located in Appendix A.

Figure 11: B Stability Monument

Figure 12: C Stability Monument

For each benchmark, overhead obstructions were measured with a clinometer and compass. These obstructions were sketched on NGS visibility obstruction diagrams. Copies of each mark’s visibility diagrams are included in Appendix B. General findings concerning monument stability and overhead obstructions and their relative impacts on accuracy will be discussed in the results section of this thesis.

For this study, some marks were not listed in the NGS database. For example, Station PEAK was set during the survey campaign. PEAK is a 2.5

inch diameter by 30 inch long stainless steel pipe solidly set 26 inches deep in rocky, mountainous ground. It is believed to be resting on top of bedrock. Figure 13 below depicts an image of PEAK. A mound of rocks were placed around PEAK for easy identification in the field. Station PEAV is also not in the NGS database, and was set by the Oregon State University College of Forestry at an unknown date. Since these marks did not follow NGS guidelines for C stability, they are both considered D stability, although PEAK is believed to be very stable. Station Q388-RESET was reset by the Benton County Surveyor, and is not formally in the NGS IDB. However, Q388-RESET should be considered as a C stability mark.

Figure 13: Station PEAK Set During This Survey

3.4 SESSIONS

Once the marks to be surveyed were selected and all of the network links from NSRS-Primary-Secondary-Local were identified, the planning phase moved to session planning. A key to this processes was to ensure that the hierarchy of the survey was followed according to NGS 58 (i.e., NSRS-Primary and Primary-Secondary-Local were all observed with the required baseline links). The NSRS-Primary baselines were relatively easy to plan considering that only five hour sessions for three different days was required to establish these baselines. The other sessions needed to be planned in an efficient manner such that all required baselines were collected while minimizing the time spent in the field.

The triviality of baselines was considered, and it was ensured that each session did not contain trivial vectors. Lines can be selected as independent or trivial. Baselines that are utilized in a survey are known as independent vectors, and rules exist on how many possible independent baselines are available for post-processing per session. Trivial baselines, also called dependent baselines, cannot be used in a single session (Van Sickle, 2008). The number of possible independent baselines is determined by eq 3, and the total number of baselines is determined by eq 4.

$$\text{\# Independent Baselines} = N - 1 \text{ ; } N = \text{\# of Receivers} \qquad \text{(eq. 3)}$$

$$\text{\# Total Baselines} = \frac{N(N-1)}{2} \text{ ; } N = \text{\# of Receivers} \qquad \text{(eq. 4)}$$

For example, if 5 receivers are used then only a maximum of 4 baselines are independent and can be used for post-processing. Figure 14 is an example of this concept with solid lines representing independent vectors and dotted lines representing trivial vectors. Additionally, all baselines must be connected to each other and no closed loops may exist using independent baselines for a single session.

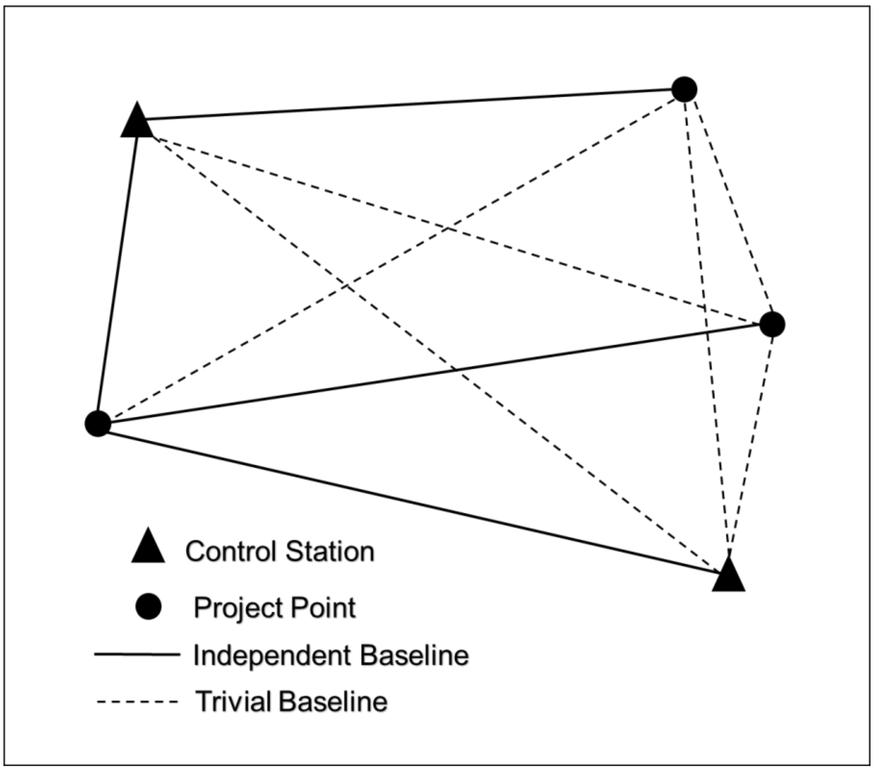

Figure 14: Independent Vector Example for a Single GNSS Session

In addition, plans were made to ensure every Secondary and Local baseline was observed at least twice at different times of the day according to requirements in NGS 58. An example diagram of one of the primary sessions portraying independent baselines is depicted in Figure 15. Also a complete listing of all sessions and stations observed in this study are presented in Table 3.

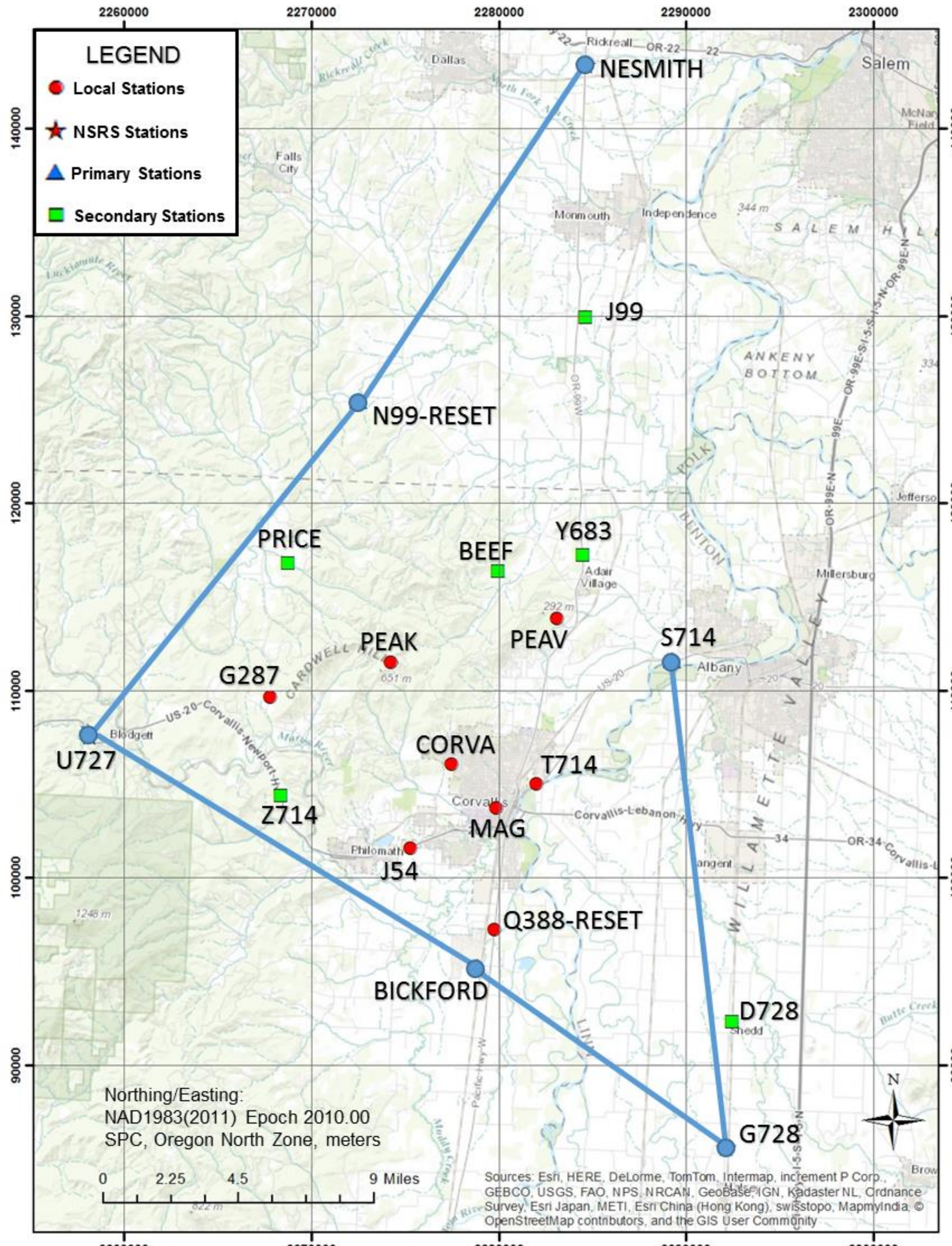

Figure 15: Sample Observation Session with Independent Vectors

Table 3: Sessions and Observations

Session	Date	U727	G728	NESMITH	BICKFORD	S714	N99-RESET	J99	Y683	BEEF	PRICE	Z714	D728	G287	J54	Q388-RESET	MAG	T714	PEAV	CORVA	PEAK
1A	14-Jul	X	X	X	X	X	X														
2A	15-Jul	X	X	X	X	X	X														
2B	15-Jul			X		X	X	X	X												
2C	15-Jul					X	X	X	X												
3A	16-Jul			X		X	X	X	X												
3B	16-Jul			X		X	X	X	X												
3C	16-Jul	X	X	X	X	X	X														
4A	22-Jul		X		X	X							X					X			
4B	22-Jul					X			X	X							X		X		
4C	22-Jul								X	X							X	X	X		
4D	22-Jul						X		X	X	X						X				
5A	24-Jul		X		X	X							X				X				
6A	25-Jul					X				X							X	X	X		
6B	25-Jul								X	X							X	X	X		
6C	25-Jul						X			X	X						X		X		
7A	29-Jul				X								X		X	X	X				
7B	29-Jul											X			X	X	X			X	
7C	29-Jul										X	X		X				X		X	
7D	29-Jul										X			X			X	X			
7E	29-Jul	X									X	X		X			X				
8A	30-Jul										X	X		X				X		X	
8B	30-Jul	X									X	X		X			X				
8C	30-Jul				X							X			X	X	X				
8D	30-Jul				X							X			X	X	X				
8E	30-Jul											X		X		X	X			X	
9A	31-Jul									X	X			X						X	X
10A	1-Aug									X	X			X						X	X
10B	1-Aug											X					X	X		X	

After all of the session requirements were identified, the actual plan for session execution and movement of the GPS receivers between sessions was planned. The predicted VDOP for each STA was also examined for each session to ensure that the NGS requirement for VDOP below 6 was not surpassed. The predicted VDOP was determined utilizing the most recent almanac and a GPS Planning tool within Leica Geo Office (LGO©). This tool allows users to upload the predicted satellite positions via the most recent almanac and combines user inputted overhead obstructions at a given position. The tool then calculates the predicted VDOP levels and time windows for any specified day.

3.5 FINAL NETWORK LAYOUT

Figures 16, 17, & 18, depict the primary network, the secondary and local networks, and the final overall network respectively. The final overall network depicts all baselines collected for this survey. These final baselines were then utilized in the post-processing adjustment for all station coordinates.

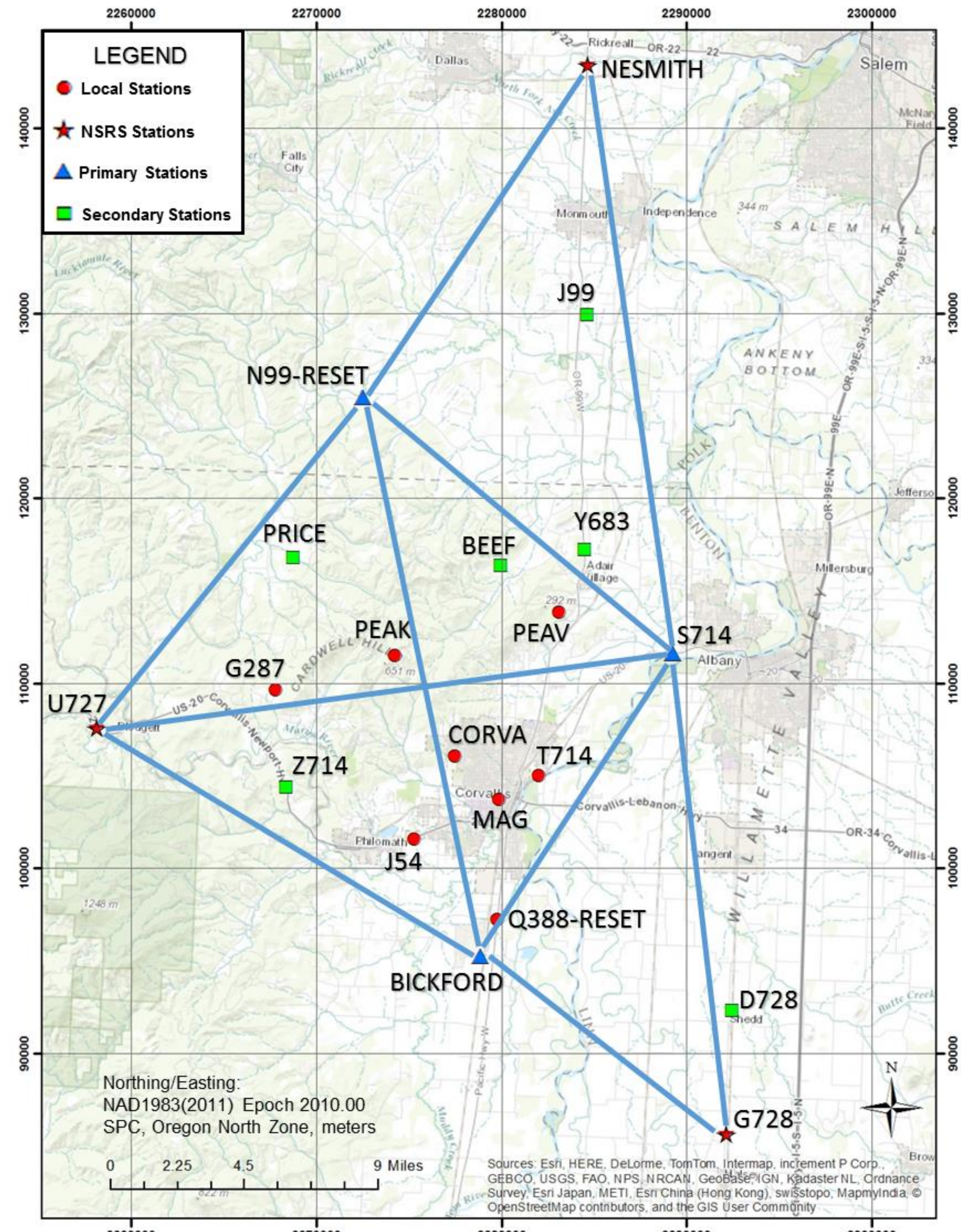

Figure 16: Primary Network Map

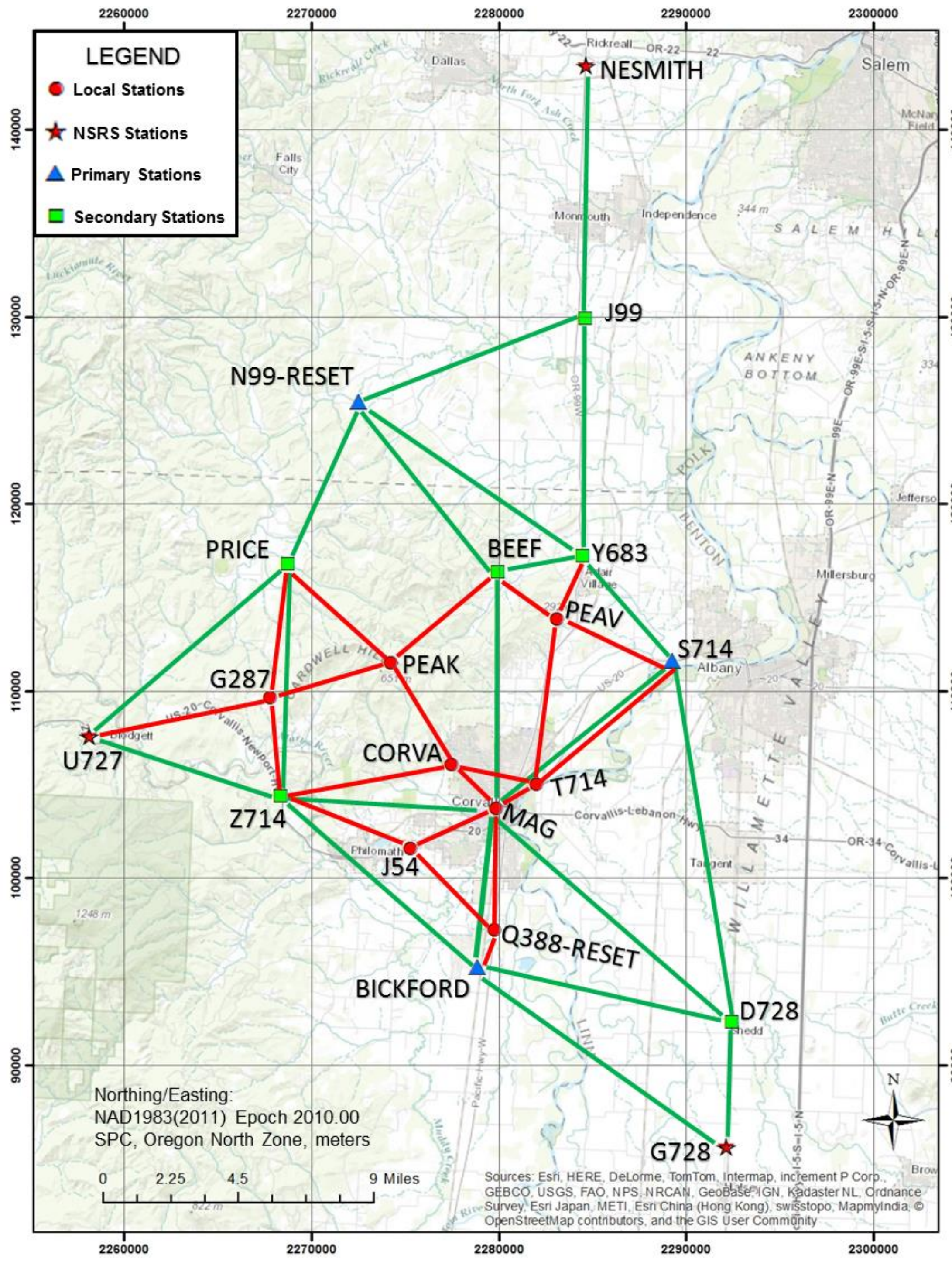

Figure 17: Secondary and Local Network Map

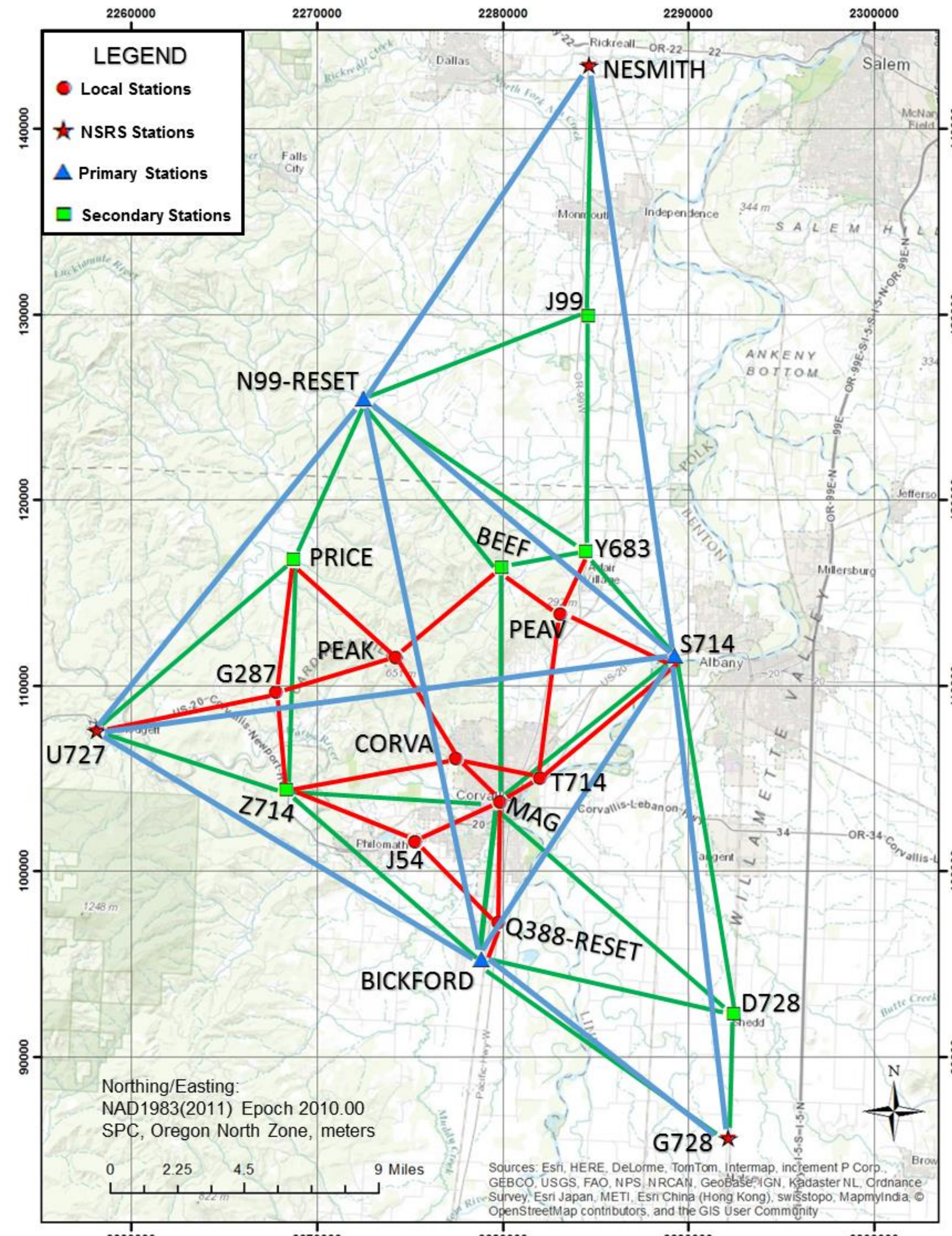

Figure 18: Complete Network Map

3.6 Survey Execution / Recommendations for Improvement

The planned survey campaign covering an area over 350 square miles was executed and completed in a total of 10 working days. The survey was conducted by five different individuals and required 28 different 1 hour or greater distinct sessions. Considering time for transportation, setup, movement, observation, and recovery, the survey fieldwork totaled 230 combined person hours. In the end, 103 independent baselines were observed prior to analyzing the data for blunders and statistical outliers.

The data was checked daily at the end of all of the sessions by uploading all of the RINEX files onto LGO and ensuring that data spanned the required time periods. Baselines were also processed as a preliminary check to determine if any problems occurred that may require future observations. If a fixed solution was not achieved for a particular baseline, then plans were made to measure the line again.

During one session, one of the GS-14 receivers failed to record signals from half of the available GPS satellite vehicles for an unknown reason. The exact cause of this anomaly is unknown, and any baselines connected to that station for that session were not used in the final network processing. This event also resulted in observing the lost baselines at a later planned date in order to ensure NGS 58 requirements were met.

One clear blunder was discovered after post-processing and will be highlighted in greater detail later in the thesis. The most likely cause of the blunder was that the antenna most likely came out of level during the observation. In order to reduce this type of blunder, it is recommended that field crews always carefully double check the plumb of their fixed-height tripod before, during, and after a session.

3.7 ADHERENCE TO NGS 58 & 59

Most of the guidelines in NGS 58 and 59 were followed with only some minor exceptions. This section will only describe the items that were not followed. Although not an explicit requirement, NGS recommended that all antennas be identical and that choke ring antennas be used to reduce the effects of multipath. For this survey, choke ring antennas were not utilized due to costs and all stations except for one were observed by the same type of receiver. The different receiver was added to ensure that all of the NSRS and Primary Stations were observed simultaneously in order to reduce the total fieldwork required. The absolute phase center variation model for that one different receiver (i.e., a Trimble R8 Model 2 set on NESMITH) was used during post-processing.

Another difference was that instead of only collecting the minimum 30 to 45 minutes of data during a session for secondary and/or local baselines, 1 hour sessions were performed. Although this added a bit more time in the field, 1 hour sessions were done for simplicity in the session scheduling and for ensuring an adequate amount of quality data was collected for each baseline.

Collecting an extra 15 minutes of data seemed prudent, as returning to the field to collect more data will be much more costly than performing somewhat longer sessions.

The requirement for taking photographs for every single session and station was not strictly followed. For this survey, close-up photos of every station, horizon photos, and photos of tripod setups were recorded for every station, but are not available for every single setup. This requirement is more for the purpose of bluebooking data, and was not necessary for research purposes.

One last requirement not followed was the NGS post-processing requirement for RMS and repeat baselines. NGS requires that the RMS values for each computed baseline must not exceed 1.5 cm and that repeat baselines that are greater than 2.0 cm difference in ellipsoid height must be re-observed (Zilkoski et al., 1997). These requirements were viewed as too restricted given that if this policy was followed, then too many baselines would be considered invalid. For instance, after removing the baselines of a clear blunder, a total of 28.4% of the baselines would not meet this criteria set above. This could be an indication of poor data or malfunctioning receivers, but it is much more likely that the random errors in GPS observations are typically greater than the NGS thresholds. It is critical to not remove random error, or one will underestimate the total uncertainty of the survey results. Therefore, a different approach to detect bad baselines was utilized by conducting a 3 sigma outlier test and then reprocessing and re-conducting a 3 sigma test in order to exclude bad baselines. More details on the specifics that were taken in removing bad baselines in this survey will be discussed later, but this requirement set forth by NGS for < 1.5 cm RMS and < 2.0 cm Up repeat baselines seems overly restrictive. The results will show that even by keeping most of the baselines that exceeded these NGS requirements in the network, the ellipsoid heights were still determined to be less than ± 2 cm at 95% confidence.

4 DETERMINING HEIGHTS FOLLOWING NGS 58 & 59

4.1 DETERMINING ELLIPSOIDAL HEIGHTS

Final baseline processing was performed once the IGS final precise ephemerides came available. IGS final ephemerides are generally available 12 days after baselines are surveyed. The scheme for post-processing included utilizing two different commercial software packages. First, the RINEX files were uploaded into Leica Geo Office (LGO©) for baseline processing. Independent, fixed baselines processed in LGO were then exported into MicroSurvey STAR*NET© for performing network least squares adjustments. More detail in determining ellipsoidal heights during post-processing is discussed below.

4.1.1 DEVELOPMENT OF NON-TRIVIAL GPS BASELINES

LGO was used to process baselines between stations. In order to process baselines according to NGS guidelines and in order to reduce orbit errors, the precise ephemerides were downloaded from IGS and uploaded into LGO. Also, the NGS absolute antenna phase center variation models for the Leica GS-14 and the Trimble R8-Model 2 were inputted. Upon uploading the RINEX data from each session, LGO displays all of the observation times at each station. Several steps were then taken prior to processing the baselines.

First, the windowing tool within LGO was used to select the appropriate interval of time where all receivers were simultaneously collecting data for each session. This manual windowing was done to ensure that the start and end time of each data file was the same for each session.

Next, several parameters were preset for processing. In order to comply with NGS 58 guidance, the elevation mask was set to 15 degrees. The corrections for frequency, Ionospheric model, and Ionospheric activity were all set to automatic which allows LGOs processing software to model these errors and apply corrections. Lastly, the Saastamoinen model was set for the tropospheric model and was selected based on other research (Brunner F, and W. Welsch, 1993). Only the GPS data was used for baseline processing.

After the parameters were set in LGO, all possible baselines were processed for each session. A quick check to ensure all baselines received a fixed solution was conducted. Lastly, the fixed, nontrivial baselines were then converted to ASCII files and uploaded into STAR*NET for pre-adjustment analysis and least squares adjustments.

4.1.2 Initial Ellipsoidal Network Processing

Once the GPS data was uploaded into STAR*NET, it is prudent to check the network for bad baselines and blunders. The methods used for checking for outliers included analyzing loop misclosures, the precision of repeat baselines, and the overall RMS of the post-adjusted baseline components. A minimally constrained least squares adjustment was made by constraining the network to the published position of one of the NSRS control stations (G728) in order to look at the adjusted residuals of each baseline in N, E, and U, and the differences in the components of the repeat baselines. Loop misclosures were conducted in order to help identify any bad baselines.

4.1.3 Blunder Detection

Repeat baseline component differences in N, E, and U were plotted in Figure 19. This figure also shows the square root of the sum of the squares (RMS) of these baseline component differences. A clear spike in RMS can also be seen in Figure 19 for one repeat baseline. Moreover, this baseline had significant error in both the northing and the easting components, which is a clue that the antenna may have not been level when one of the repeat baselines was observed. After further evaluation of the two sessions associated with this spike, it was determined that all trivial baselines coming into station BICKFORD during session 4A had large differences in their components when compared with other repeat baselines. It appears that the receiver must have come out of level on BICKFORD during session 4A, and any baselines connected to this station during that session were removed from the adjustment. These baselines were also ignored when detecting potential statistical outliers in the data.

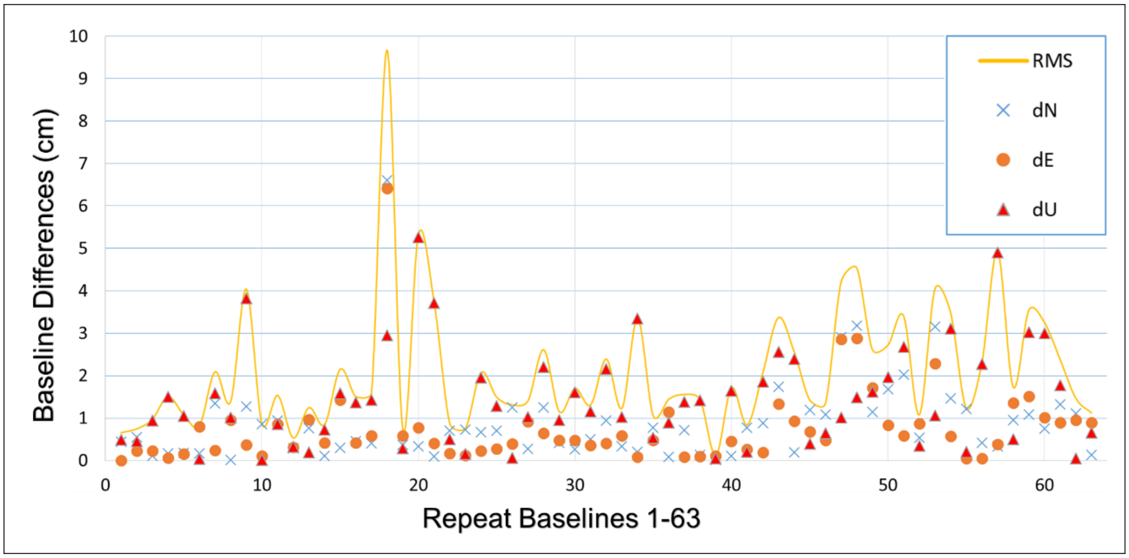

Figure 19: Initial RMS Plot of Repeat Baseline Differences

4.1.4 Statistical Outlier Removal

Analysis of repeat baselines and their differences between each other in regards to the northing, easting, and up components is one technique recommended by NGS to identify problems in the survey data. In theory, repeat baselines should have relatively the same change in the northing, easting, and up and should not have significant differences in those components. Since NGS's threshold for repeat baselines was deemed too restrictive, a new outlier test was created and excluded baselines beyond 3 standard deviations from the mean repeat baseline difference of the RMS for northing, easting, and up.

After an initial 3 sigma test, 7 out of the 62 repeat baselines failed to meet the standards. The repeat baselines were analyzed in detail and four out of the seven outlying repeat baselines were excluded as clear outliers. A problem developed in the other three, because the network only contained two baselines for those respective repeat baselines. Therefore, in order to determine which baseline was actually bad, loop misclosure analysis was conducted. After conducting the loop misclosure analysis, three more baselines were identified as outliers and removed. Figure 20 shows the repeat baselines after the blunder and seven outliers were removed. Further refinement of repeat baselines was not conducted and the method for identifying outliers shifted to residual analysis.

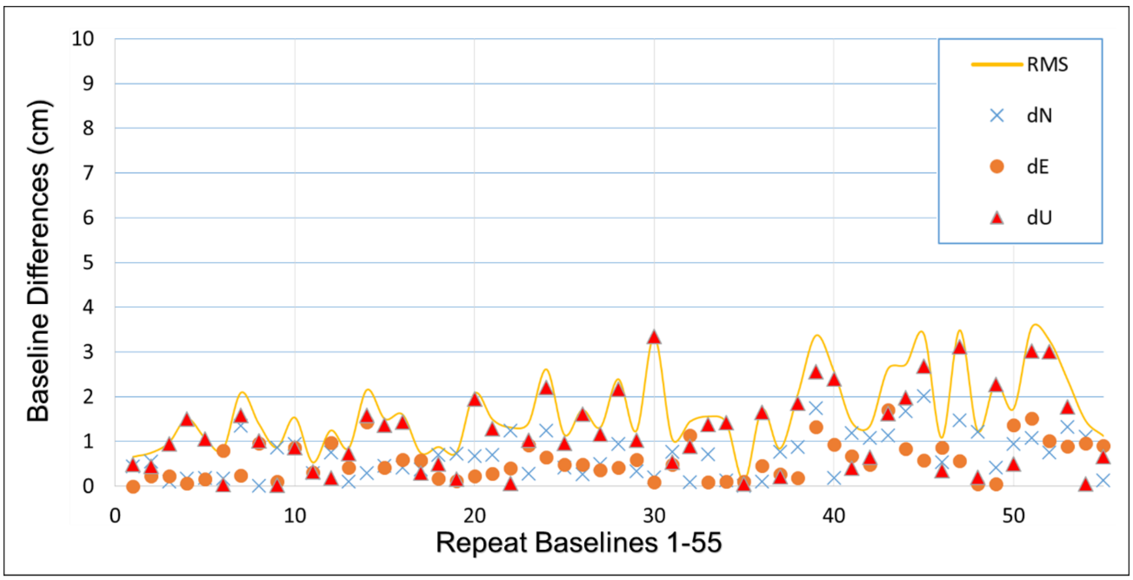

Figure 20: Repeat Baseline Differences with Outliers Removed

Next, the residuals in N, E, and U were examined for all independent baselines within the network. The initial free adjustment network was conducted again with the blunder and previous outliers removed in order to produce a new set of residuals. Using the residuals on this second adjustment, the residuals in northing (Figure 21) had none of the baselines exceed 2 cm and the average residual was only 4.8 mm. For residuals in easting (Figure 22), only 1 baseline exceed 2 cm which represented 0.97% of the data, and the average residual was 3.5 mm. This baseline was deemed an outlier and was removed. For residuals in the up component (Figure 23), 7.77% exceed 2 cm, and the average residual was 9.2 mm. Baselines that exceeded 3 cm in residual, were determined as 3 sigma outliers, and were removed.

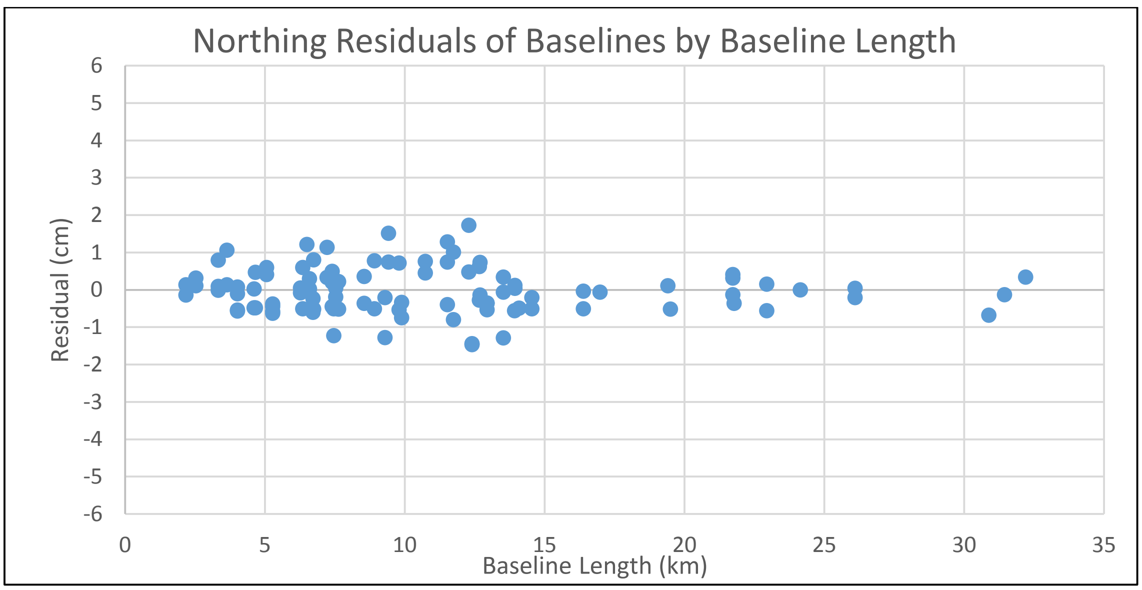

Figure 21: Northing Residuals of Baselines by Baseline Length

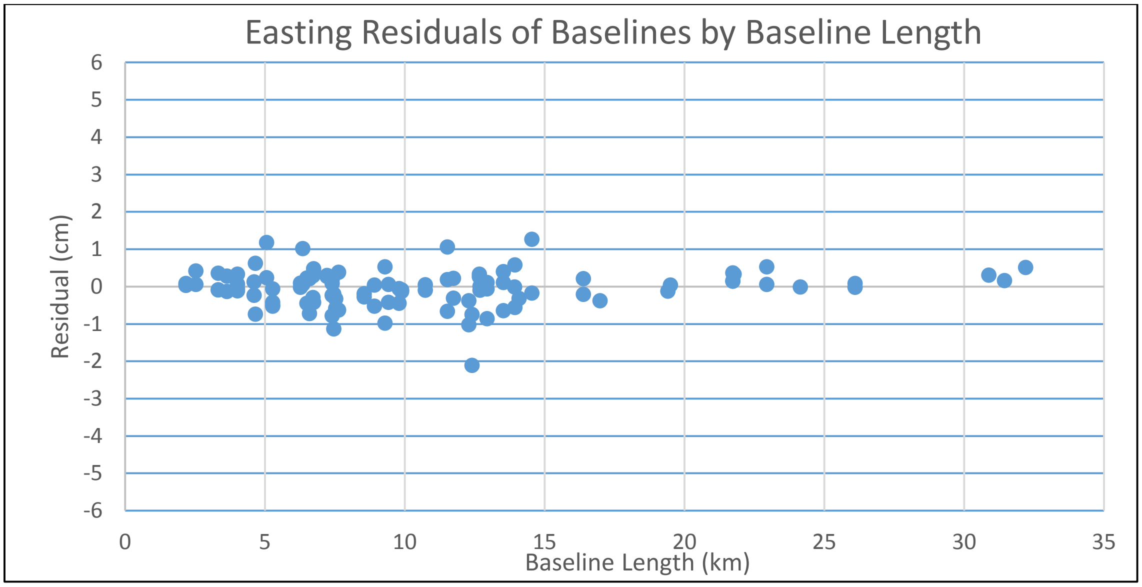

Figure 22: Easting Residuals of Baselines by Baseline Length

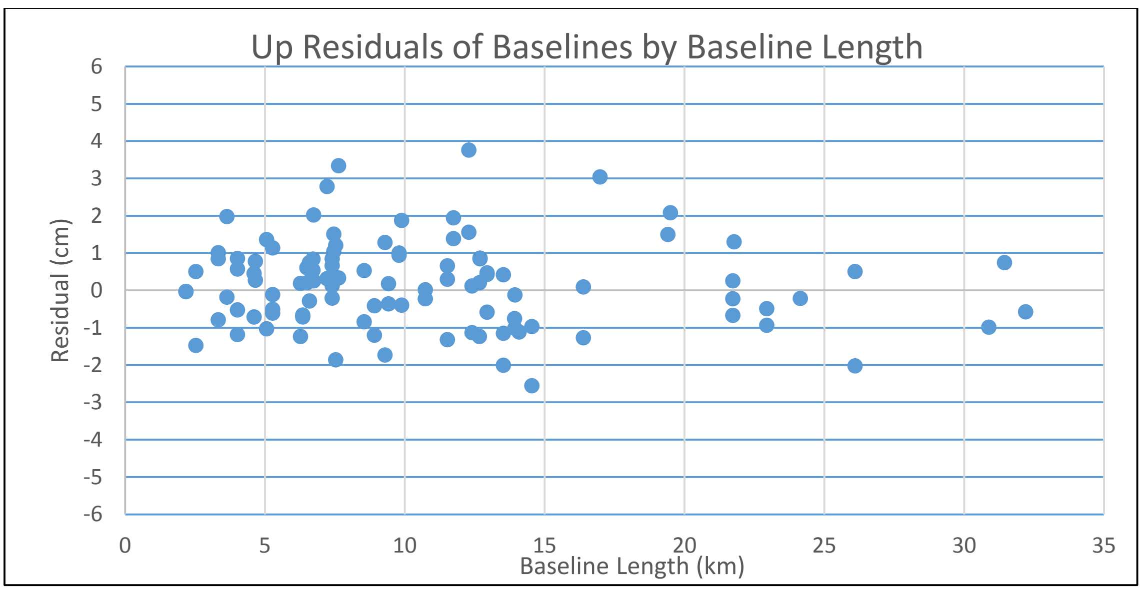

Figure 23: Up Residuals of Baselines by Baseline Length

The horizontal residuals were about twice as precise as the vertical residuals. This is to be expected since the horizontal component within GPS surveying is generally about twice as accurate due to the geometry of satellites overhead and the lack of satellites from underneath. As a result, the trilaterated solution produces more precise and accurate horizontal solution compared to the vertical solution.

The network was then adjusted again, and the residuals for northing, easting, and up were re-checked. After removing all of the previous outliers, the residuals now all passed the 3 sigma test of their respective component residuals. In fact, northing (Figure 24) and easting (Figure 25) were both under 2 cm in residual and up (Figure 26) contained only 5.05% above 2 cm and 2.02% above 2.5 cm. At this point, it was determined that all statistically outlying baselines had been removed from the network.

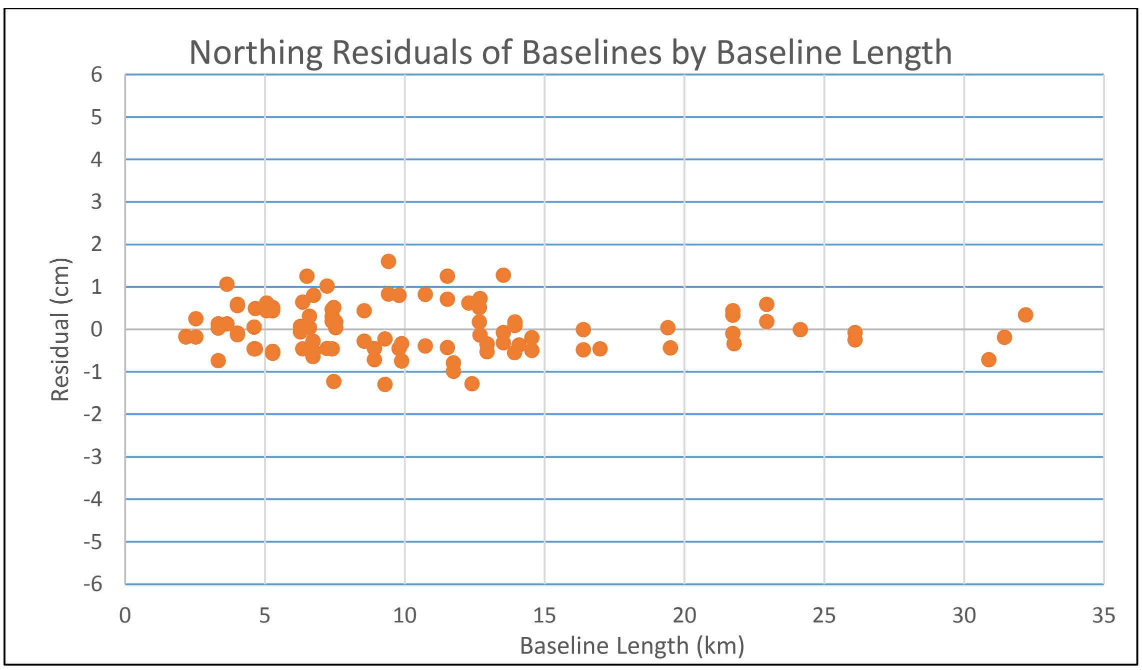

Figure 24: Northing Residuals with Outliers Removed

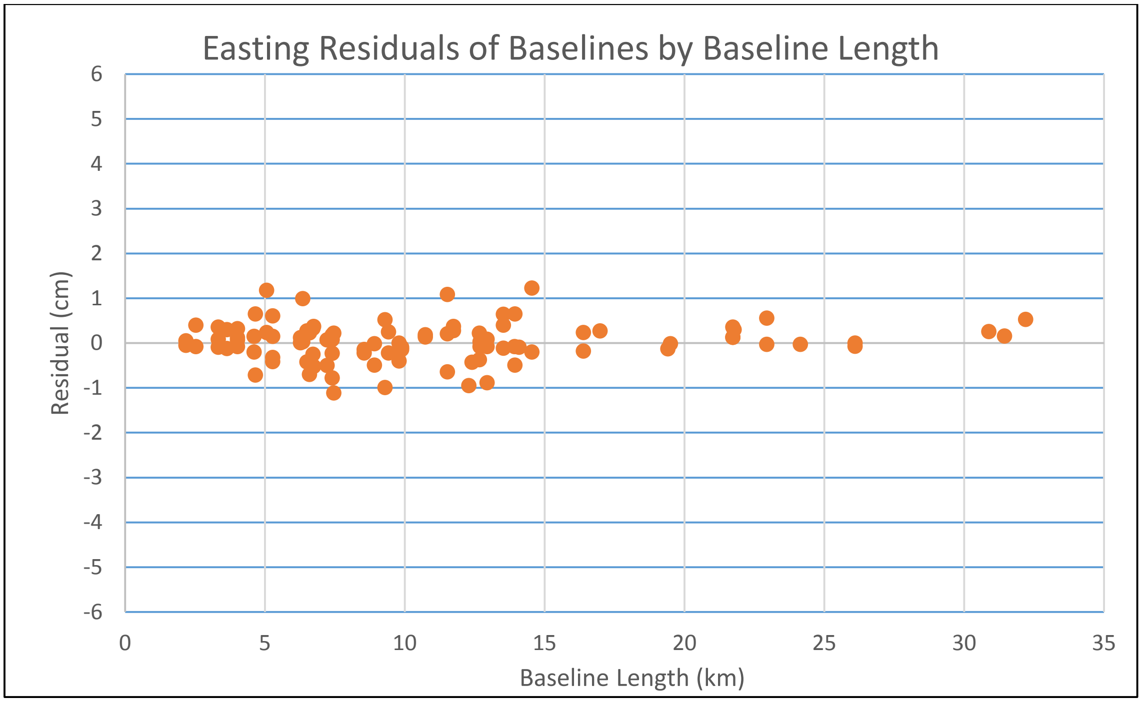

Figure 25: Easting Residuals with Outliers Removed

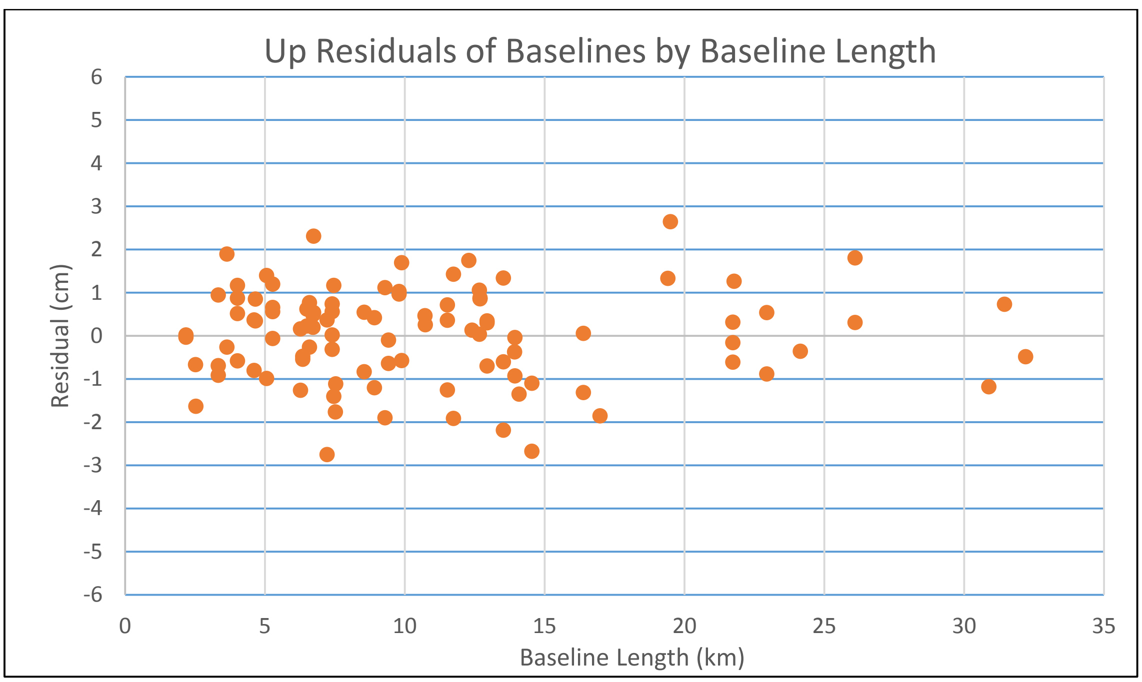

Figure 26: Up Residuals with Outliers Removed

Every time a baseline was identified as a candidate for removal due to being an outlier, the individual session containing that baseline was re-examined in order to verify the non-triviality of the session network design. In some cases, an additional baseline that was not identified as being independent in the planning phase was added due to the loss of the outlier. This ensured that as many baselines were included into the overall network as possible while maintaining non-triviality and complete links of all required baselines. Moreover, baselines that were outliers were checked to ensure that every required connection identified during the planning phase was still connected by at least one baseline.

For every subsequent least squares adjustment, the process explained above of checking residuals for outliers exceeding 3 standard deviations from the mean was repeated. At no step beyond this initial scrubbing for outliers was a baseline later identified as an outlier. This checking for outliers was reiterated in order to ensure that no bad baselines influenced the final results of the ellipsoidal and orthometric processing.

4.1.5 Scaling Criteria and Constraints

The next step in the post-processing workflow was to ensure that the stochastic model of the least squares adjustment was both realistic and sufficient to pass the chi-square test. During the initial adjustments used to identify blunders, the stochastic model or weighting of the adjustment was left at

STAR*NET defaults and the results failed to pass the chi-square test. Once all outliers were removed and the adjustment was weighted properly, then the final error ellipses and 95% confidence windows would be statistically accurate.

First, centering errors were estimated. The errors used were 2 mm horizontal and 4 mm vertical. These values fall into the range of real world centering errors for most GPS platforms and also are within the range of normal instrument centering for traditional geodetic leveling (Ghilani, 2010). Next, the standard error factors were altered in order to force the adjustment to pass the chi-square test. Knowing that vertical errors in GPS are usually 1.5 to 2 times greater than horizontal errors, the vertical error scale factor was multiplied twice the factor value of the horizontal error scale factor. After testing, the final scale factor that was used was 21.00 horizontal and 42.00 vertical. The resulting chi-square test past and resulted in a standard error of unit weight of 1.000.

The final key step prior to conducting the final least squares adjustment was selecting the proper control as a constraint. If a control mark with inaccurate coordinates is used, then the network adjustment will contain the errors of the control. One of the Primary marks or one of the NSRS station marks were preferred for control due to their longer observations. To help decide which mark to hold as control, a series of minimally constrained least squares adjustments were performed, holding each one of the six stations tabulated in Table 4. Table 4 shows the average magnitude that all of the stations in the network were adjusted (in northing, easting, and up), according to which station was held as control in the minimally constrained least squares adjustment. The table also lists the range of which all stations were adjusted in the network.

Table 4: Constraint Testing Results

STA	Average Adj. Northing (cm)	Average Adj. Easting (cm)	Average Adj. Up (cm)	Adjustment Range Easting (cm)	Adjustment Range Northing (cm)	Adjustment Range Up (cm)
U727	-0.7	-0.4	-1.9	-5.5 to +1.7	-6.2 to +1.8	-4.0 to +0.1
G728	0.4	-0.1	-0.1	-4.5 to +2.7	-5.9 to +2.0	-2.2 to +1.9
NESMITH	-0.1	-0.8	-1.2	-4.9 to +2.3	-6.6 to +1.4	-1.0 to +3.2
BICKFORD	0.5	-0.2	0.7	-4.4 to +2.8	-5.9 to +2.0	-1.4 to +2.7
S714	0.6	-0.6	1.8	-4.2 to +2.9	-6.3 to +1.6	-4.0 to +0.2
N99RESET	-0.5	-1.2	2.2	-5.4 to +1.8	-6.9 to +1.0	-0.0 to +4.1

For this study, station BICKFORD was identified as the control station that would be held fixed for the ellipsoid adjustment. When constraining to either G728 or BICKFORD, the lowest average adjustment in the up component was obtained, and on average, the adjustments for the network were somewhat centered about zero in all three directions. Additionally, BICKFORD had the best overhead open coverage for satellite observations due its location on the Corvallis Airport. BICKFORD's reputation amongst local surveyors is also high. This station included long observations because it was a primary mark directly linked to two of the NSRS stations, and BICKFORD contained the second most number of baseline connections to other marks throughout the entire network. Lastly, BICKFORD's published coordinates matched well to another study conducted by Oregon State University's Geomatics program that incorporated CORS data.

4.1.6 Final Ellipsoidal Heights

Now that the data has been scrubbed for outliers, scaled, and constrained by valid control, a minimally constrained least squares adjustment was ran in order to derive the northing, easting, and ellipsoid heights for each station. The results of this adjustment was used for the final coordinates for each station in terms of these parameters. This adjustment also completes the post-processing following NGS 58's guidelines. Table 5 below shows the results of processing the ellipsoid and shows the adjusted ellipsoid height, the changes from the published coordinates in terms of dN, dE, and dU, and the vertical 95% confidence region.

Table 5: Results of Ellipsoidal Adjustment

Station	Adjusted Ellipsoidal Height (m)	dN Published (cm)	dE Published (cm)	dU Published (cm)	Vertical 95% Confidence (cm)
U727	204.768	1.12	0.23	2.59	1.60
G728	61.317	0.10	-0.04	0.77	1.27
NESMITH	41.514	0.53	0.65	-0.46	1.82
BICKFORD	52.329	Constrained	Constrained	Constrained	Constrained
S714	43.628	-0.15	0.37	2.53	1.22
N99RESET	53.330	0.98	0.99	-1.44	1.34
J99	53.891	0.94	-0.76	-0.66	1.89
Y683	71.995	1.21	-0.39	0.52	1.47
BEEF	120.518	0.37	0.13	2.70	1.40
PRICE	113.610	2.78	2.00	0.78	1.87
G287	179.954	0.99	0.69	1.48	1.67
J54	63.700	-4.38	-5.94	-0.11	1.79
T714	46.759	0.90	-0.03	0.71	1.67
CORVA	103.478	0.89	-0.48	0.70	1.80
D728	56.099	N/A	N/A	N/A	1.81
MAG	47.421	N/A	N/A	N/A	1.29
PEAV	106.054	N/A	N/A	N/A	1.62
Z714	148.861	N/A	N/A	N/A	1.50
Q388RESET	51.906	N/A	N/A	N/A	1.18
PEAK	585.359	N/A	N/A	N/A	1.91

According to the results of this part of the processing, the study meets the 2 cm standard. The average vertical error at 95% confidence was 1.59 cm with a maximum of 1.91 cm, and the largest semi-major axis for a horizontal error ellipse was 7.1 mm. Stations PRICE and J54 depicted large differences from published. Station PRICE was just beyond 2 cm from published at 2.8 cm and 2.0 cm residual in northing and easting, but Station J54 portrayed a major change from published at 4.4 cm and 5.9 cm, respectively. In fact, this station appears to have been disturbed, because the concrete block that the monument is set in seems out of level. This mark was observed in order to see if it was possible to identify benchmark movement since the date the coordinates were published. The network was never constrained to this mark, and the fact that it was out of position did not affect the overall results. A close up photo showing the potential movement of this station are depicted below in Figures 27 and 28. As expected, the adjusted coordinates on this station significantly differed from the published coordinates in the NGS IDB.

Figure 27: Setup on Station J54

Figure 28: Station J54 (note tilt of station out of level)

Furthermore, this station lies in a median island of lower elevation surrounded by roadway, and during storms this area floods causing the soil to shift. Whatever the case for the movement may ultimately be, this station appears both visibly disturbed and the survey results support this assumption. For other significant changes from published, three marks were also adjusted greater than 2 cm in ellipsoid height and include stations U727, S714, and BEEF. These published discrepancies could be due to mark movement or possibly bad data. These three stations also had medium to low overhead visibility which may have contributed to problems.

4.2 DETERMINING ORTHOMETRIC HEIGHTS

The process for determining orthometric heights as depicted in NGS 59 and explained earlier was followed. First, a minimally constrained orthometric adjustment was made in order to identify which marks are considered valid. The network was checked for any possible tilted planes in the local geoid and adjusted accordingly. Then, the elevation of all valid benchmarks were held fixed along with one northing and easting. After the fully constrained adjustment, the data was checked for over constraint and was reprocessed if necessary.

4.2.1 MINIMALLY CONSTRAINED ORTHOMETRIC ADJUSTMENT

For the minimally and fully constrained orthometric adjustments, the same parameters and baselines previously used for processing the ellipsoid were used. Residuals were continuously analyzed using the 3 sigma test, but at no time did new outliers occur. For orthometric control, station CORVA was used as the initial elevation to be fixed based on its centrality to the network. Moreover, the difference between CORVA's published elevation and the calculated elevation using the previously derived ellipsoid height and applying a GEOID12A separation correction was one of the smallest of all stations. Station BICKFORD was also used to fix latitude and longitude for the minimally constrained adjustment. The resulting marks that fell within 2 cm from the published value after this adjustment are listed in Table 7 in the section 4.2.4 Valid Orthometric Marks below.

4.2.2 CHECKING FOR GEOID TILT

Since the overall area of the network surpasses 50 km by 50 km, the area was checked for any indication of a tilt to the geoid as mentioned in NGS 59 (Zilkoski, et., al, 2008). This was done visually by investigating any trends in the changes from the minimal constrained adjustment via a spatial layout. The map in Figure 29 below shows all stations and the associated adjustment cm change in elevation from the published values.

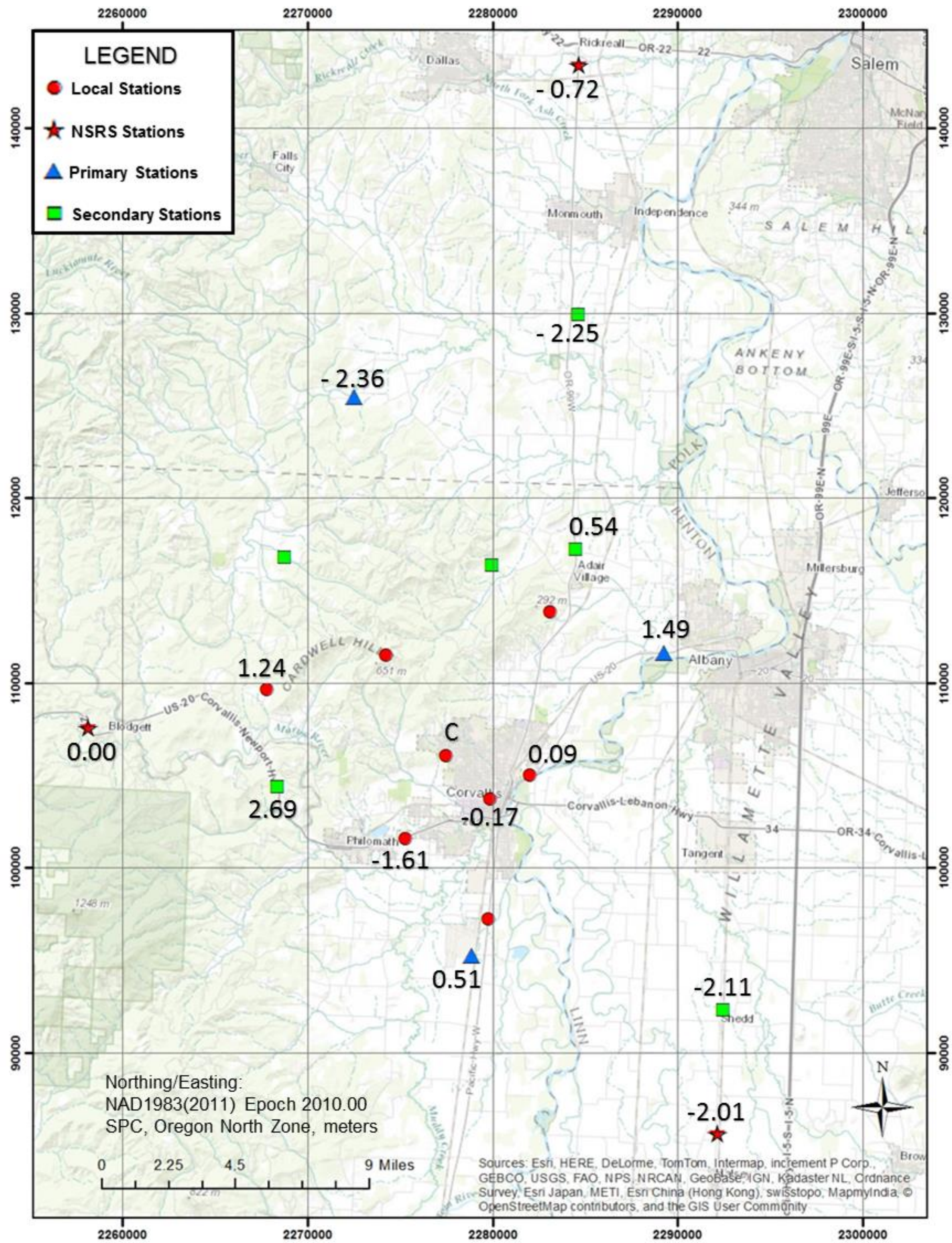

Figure 29: Minimally Constrained Height Changes from Published (cm)

After viewing the changes, no systematic trend or tilt was detected. Therefore, no adjustment for a tilted geoid plane was entered into the subsequent adjustments.

4.2.3 GEOID12A Prediction Analysis

A practice that is often used by surveyors is to only utilize the latest geoid model and apply a correction to ellipsoid heights that are found in the field either from OPUS observations or from more complex static surveys. This geoid prediction method was used to see what the results should be if the geoid model is correct. Applying a geoid correction to the ellipsoid heights derived from the NGS 58 process was conducted in order to compare the final orthometric results with ones determined using this technique. These results will later demonstrate the accuracy of the geoid model for the project area and help determine if any published values contain additional error. The resulting orthometric heights derived solely by applying a geoid correction calculation using the GEOID12A model and eq. 2 are presented in Table 6 below.

Table 6: GEOID12A Elevation Prediction

Station	Project h_{83} (m)	Modeled N_{12A} (m)	Predicted Elevation ($H_{88}=h_{83}-N_{12A}$) (m)	Published H_{88} (m)	Delta Elevation (cm)
U727	204.768	-22.023	226.790	226.783	-0.74
G728	61.317	-23.092	84.408	84.421	1.28
NESMITH	41.514	-22.409	63.923	63.923	-0.02
BICKFORD	52.329	-22.755	75.084	75.072	-1.24
S714	43.628	-22.858	66.486	66.464	***-2.22***
N99RESET	53.330	-22.084	75.414	75.43	1.63
J99	53.891	-22.535	76.427	76.442	1.52
Y683	71.995	-22.618	94.613	94.600	-1.27
G287	179.954	-22.142	202.096	202.076	-1.98
J54	63.700	-22.576	86.276	86.285	0.87
T714	46.759	-22.730	69.489	69.481	-0.82
CORVA	103.478	-22.546	126.024	126.017	-0.73
D728	56.099	-23.080	79.179	79.193	1.37
MAG	47.421	-22.681	70.102	70.096	-0.57
Z714	148.861	-22.300	171.160	171.126	***-3.42***

Out of all the marks with published elevations, only two out of the fifteen were greater than 2 cm from their published elevations. These results may have occurred, because most of the marks used in this survey were included in the development of GEOID12A. The two marks that are greater than 2 cm were S714 and Z714 with differences of -2.22 cm and -3.42 cm. At the conclusion of the orthometric processing steps, the GEOID12A model will be analyzed further.

4.2.4 SELECTION OF VALID ORTHOMETRIC MARKS

Using the results of the minimally constrained adjustment, all valid NAVD88 benchmarks for the network were identified and fixed. As previously mentioned in the literature review, this process is not an exact science and requires the user to make some determination on what is considered valid. NGS 59 does state that marks with elevations greater than 2 cm in difference from published when running a minimally constrained adjustment are clearly invalid. For the purposes of this study, no additional threshold smaller than 2 cm was selected to determine valid NAVD88 benchmarks, and the 2 cm rule was used as the only rule. Table 7 below shows all of the differences in elevation from the published value after running the first adjustment. All bolded marks were within 2 cm of their published elevations and were considered to be valid NAVD88 benchmarks.

Table 7: Valid Published NAVD88 Benchmarks

Station	dH from posted (cm)
U727	***0***
G728	-2.01
NESMITH	***-0.72***
BICKFORD	***0.51***
S714	***1.49***
N99RESET	-2.36
J99	-2.25
Y683	***0.54***
G287	***1.24***
J54	***-1.61***
T714	***0.09***
CORVA	***0***
Z714	2.69
D728	-2.11
MAG	***-0.17***

4.2.5 Fully Constrained Adjustment

A fully constrained adjustment holding all valid NAVD88 benchmarks fixed was then conducted. The latitude and longitude of station Bickford was also held fixed for horizontal control. The results of this adjustment are depicted in Table 8 below. For this iteration, none of the free marks with published elevations resulted in valid elevations, and their differences from published ranged from -3.01 cm to 2.38 cm.

Table 8: Fully Constrained Orthometric Results

Station	dH from published (cm)
U727	Constrained
G728	-2.97
NESMITH	Constrained
BICKFORD	Constrained
S714	Constrained
N99RESET	-2.58
J99	-2.21
Y683	Constrained
G287	Constrained
J54	Constrained
T714	Constrained
CORVA	Constrained
Z714	2.38
D728	-3.01
MAG	Constrained

This step was not the end of the orthometric adjustment. First, the changes needed to be checked for an over constraint, and if a mark was deemed to be overly constrained, the network would be adjusted again.

4.2.6 Checking Impacts of Constrained Marks

Differences between the resulting elevations from the minimally constrained and the fully constrained were calculated in order to detect an overly constrained adjustment. NGS suggests that differences between neighboring stations should generally not be greater than 1 cm, and a difference in 2 cm is a clear sign that invalid or incorrect vertical control was fixed (Zilkoski et al., 2008). A map that depicts spatially the differences between the minimally and fully constrained adjustments is depicted in Figure 30 below.

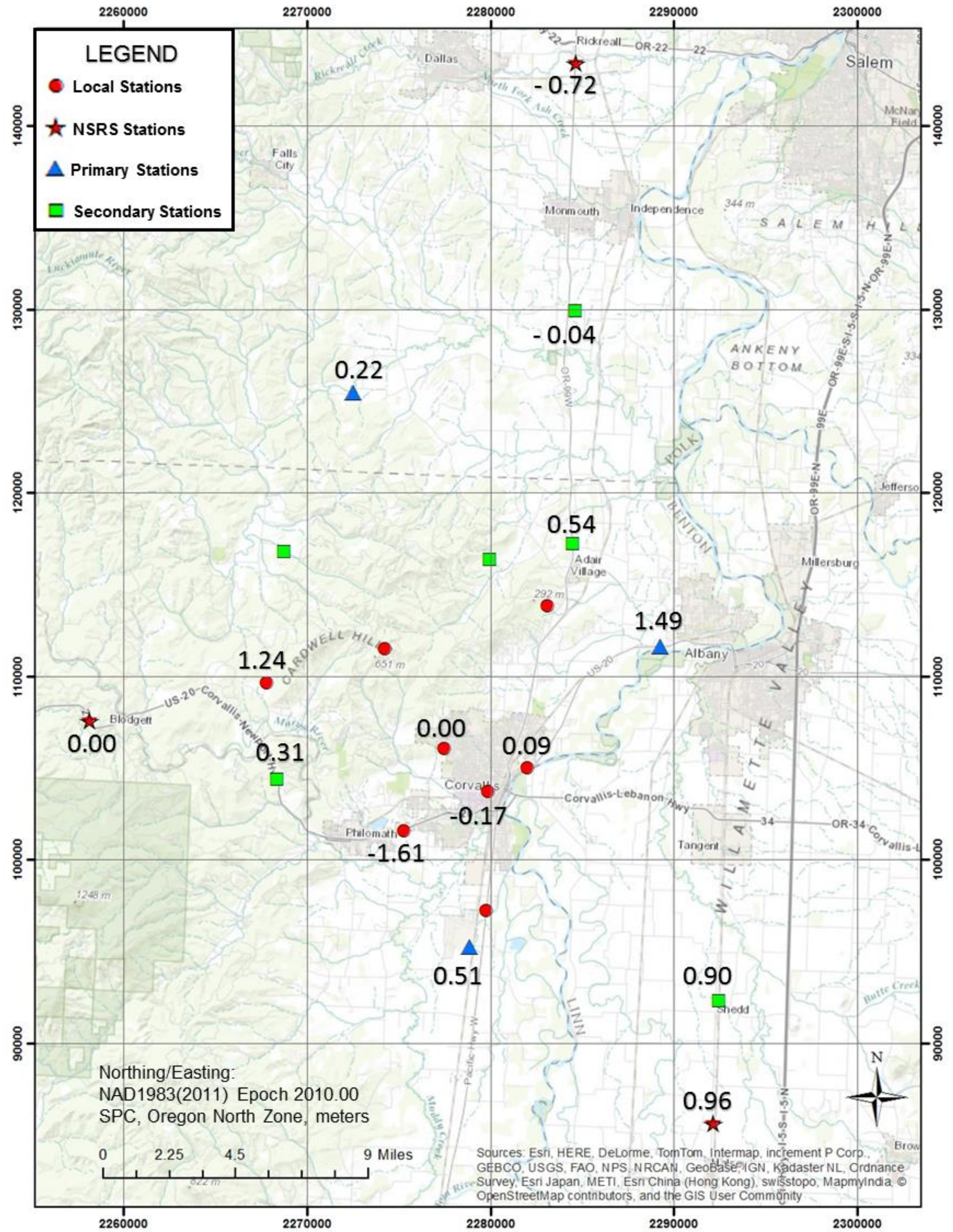

Figure 30: Differences Between Minimally and Fully Constrained (cm)

Looking at the differences, three stations appeared to show signs of over constraint. S714's difference between the two adjustments was 1.49 cm and the nearby station of T714 had a difference of only .09 cm resulting in a 1.40 cm difference between two neighbors. J54 had a difference of -1.61 cm between the two adjustments which was over a 1 cm difference between its neighbors at MAG and BICKFORD. Lastly, G287 had a difference between the adjustments of 1.24 cm which was greater than 1 cm from its neighbors of U727 and CORVA. Due to these facts, these three stations were allowed to float for another iteration of a fully constrained adjustment.

4.2.7 Final Fully Constrained Adjustment

The fully constrained adjustment was ran for a final time while allowing the overly constrained marks to float. The results of this adjustment are again compared to the minimally constrained adjustment, and the differences are displayed spatially in Figure 31 below. Now, all of the marks had adjustment differences less than 1 cm when compared to neighboring stations.

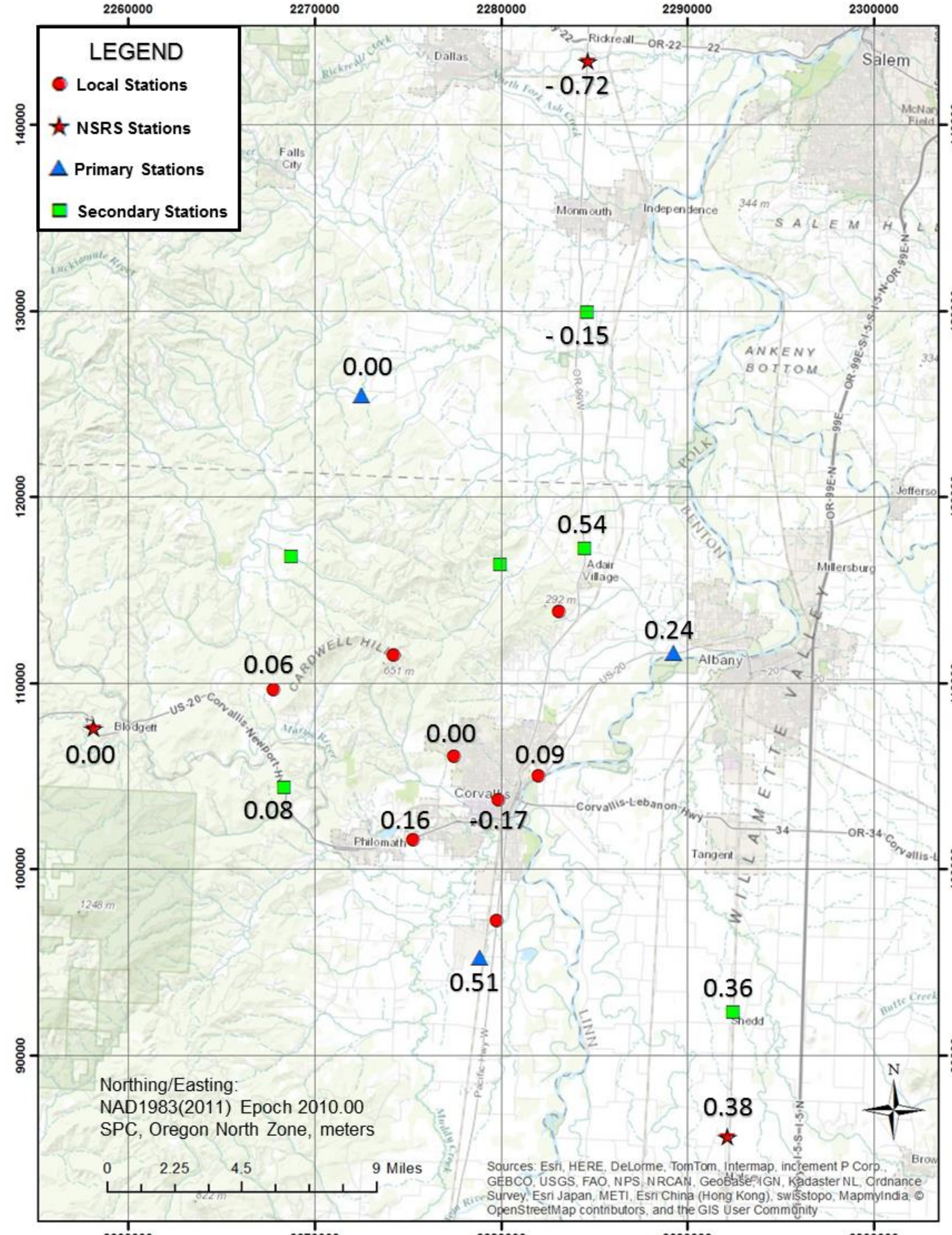

Figure 31: Final Differences Between Minimally and Fully Constrained (cm)

Figure 32 is a map of the published benchmarks and their final changes, and Table 9 below shows the elevation results from the orthometric adjustment.

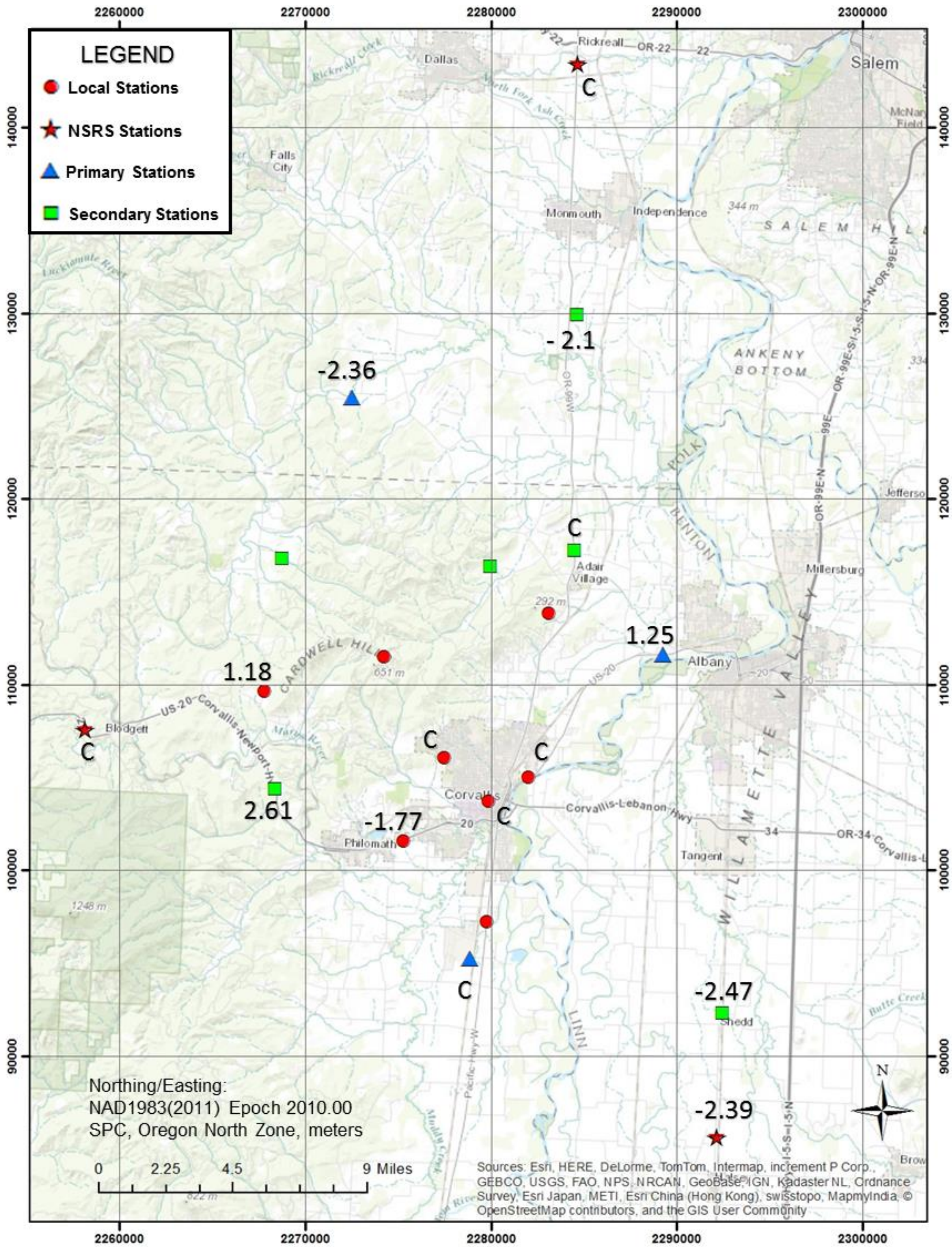

Figure 32: Final Orthometric Changes from Published (cm)

Table 9: Final Orthometric Adjustment Results

Station	Elevation (m)	Change from Published (cm)	Elevation 95% Confidence (cm)
U727	226.783	Constrained	Constrained
G728	84.397	-2.39	1.2
NESMITH	63.923	Constrained	Constrained
BICKFORD	75.072	Constrained	Constrained
S714	66.477	1.25	0.8
N99RESET	75.406	-2.36	0.9
J99	76.421	-2.1	1.4
Y683	94.600	Constrained	Constrained
BEEF	142.914	N/A	0.9
PRICE	135.645	N/A	1.6
G287	202.088	1.18	1.4
J54	86.267	-1.77	1.7
T714	69.481	Constrained	Constrained
CORVA	126.017	Constrained	Constrained
D728	79.168	-2.47	1.8
PEAV	128.639	N/A	1.2
MAG	70.096	Constrained	Constrained
Z714	171.152	2.61	1.2
Q388RESET	74.659	N/A	1.1
PEAK	607.584	N/A	1.6

Of the marks that were greater than 2 cm from their published elevations, none of the marks were greater than 3 cm. The largest difference was Z714 with a 2.61 cm difference from its published elevation. Interestingly, J99 and N99-RESET both had differences of -2.36 cm and -2.1 cm from published, and this could have been an issue with the 99 leveling line. N99-RESET was also a reset mark that could have had an erroneous updated elevation. G728 and D728 were also on each other’s level line and on the edge of the project, and they may have incorrect published elevations due errors in the 728 level line. Numerous other factors such as uplift, subsidence, and disruption may be factors on why the elevations of these marks are outside of 2 cm. Overall, the 95% confidence intervals for the marks that were allowed to float ranged from a maximum of 1.79 cm to a low of 7.9 mm with an average confidence interval of 1.29 cm.

4.2.8 EVALUATING CHANGES TO GEOID

Since the orthometric adjustment essentially constrains the topographic surface and holds the latest geoid model fixed, the error within the network was forced into the ellipsoid heights derived during the orthometric adjustment. Since the errors are forced into these values, the ellipsoid values resulting from the orthometric adjustment are usually never reported, but they may offer some utility for analysis. For instance, these values were compared to the ellipsoidal heights derived under the procedures of NGS 58 to evaluate the quality of the geoid model. Moreover, the GEOID12A height values for each mark were compared to the new geoid heights (N_{new}) derived by using equation 1 with the final ellipsoid and elevation results as inputs. These values are listed in Table 10 below.

Table 10: Local Geoid Model Comparison

Station	h_{83}(NGS59) - h_{83}(NGS58) (cm)	N_{12A} - N_{new} (cm)
U727	-0.74	-0.74
G728	-1.1	1.28
NESMITH	-0.02	-0.02
BICKFORD	-1.24	-1.24
S714	-0.97	-0.5
N99RESET	-0.73	-0.71
J99	-0.58	-0.58
Y683	-1.27	-1.27
BEEF	-0.89	-0.94
PRICE	-0.76	-0.76
G287	-0.79	-0.84
J54	-0.9	-1.05
T714	-0.82	-0.82
CORVA	-0.73	-0.73
D728	-1.1	0.5
MAG	-0.57	-0.95
PEAV	-0.94	-0.87
Z714	-0.81	-0.9
Q388RESET	-1.06	-1.15
PEAK	-0.81	-0.85

The average difference between the NGS 58 derived ellipsoidal heights and the error containing ellipsoid heights resulting from the orthometric adjustment was -0.84 cm. The largest difference was -1.27 cm at station Y683, and only 5 marks were over 1 cm. Of the five, two were included in the initially modeling of GEOID12A. All of the locally derived geoid heights from the study were all within 1.5 cm of the modeled GEOID12A values with the largest difference occurring at G728 at 1.28 cm of difference. After analyzing these geoid results, the data seem to fit the geoid rather well and is within 95% of GEOID12A's error, which is at least 4 cm in magnitude in this area (National Geodetic Survey, 2015a).

Interestingly, the final adjusted orthometric values compare closely with the previously predicted orthometric values using GEOID12A and the earlier derived ellipsoidal heights presented in Table 6 in section 4.2.3 above. A listing for this comparison is in Table 11 below and none of the values differed by more than 2 cm. Differences did occur on which benchmarks were considered valid. Following the GEOID12A calculations alone, Z714 and S714 were identified as being outside of 2cm from published, but the final orthometric processing shows S714 was within 2 cm and that G728, N99RESET, J99, Z714, and D728 were all outside 2 cm. Because the adjustment results are much more mathematically robust than the mere geoid correction, the full orthometric adjustment is recommended over the simple correction technique often used in practice.

Table 11: Final Elevations vs. Geoid Predictions

Station	**Predicted Elevation $H_{88} = h_{83(NGS58)} - N_{12A}$ (m)**	**Project Elevation $H_{88(NGS59)}$ (m)**	**Difference (cm)**
U727	226.790	226.783	-0.74
G728	84.408	84.397	-1.11
NESMITH	63.923	63.923	-0.02
BICKFORD	75.084	75.072	-1.24
S714	66.486	66.477	-0.97
N99RESET	75.414	75.406	-0.73
J99	76.427	76.421	-0.58
Y683	94.613	94.600	-1.27
G287	202.096	202.088	-0.80
J54	86.276	86.267	-0.90
T714	69.489	69.481	-0.82
CORVA	126.024	126.017	-0.73
D728	79.179	79.168	-1.10
MAG	70.102	70.096	-0.57
Z714	171.160	171.152	-0.81

5 DETERMINING HEIGHTS UTILIZING OPUS TECHNIQUES

5.1 INTRODUCTION TO OPUS AND OP

During most of the NSRS history, access to the system was obtained via a network of passive benchmarks who had their coordinates determined by extensive geodetic surveys and adjustments (Stone, 2006). Starting in 1994, NGS announced that they would no longer maintain benchmarks and implemented a series of active stations that collected GPS data continuously for 24 hours a day, 7 days a week. This network that NGS implemented was named as the Continuously Operating Reference Stations (CORS). The intent was that CORS would provide survey control in the future (Steinberg and Even-Tzur, 2008), and the network supports three-dimensional GPS positioning. This new network also allowed surveyors, free of charge, the means to actively access the NSRS through computed coordinates and velocities based on past data (Stone, 2006). As of January 2014, CORS is expanding and contained more than 1,900 stations that were contributed by over 200 different organizations (National Geodetic Survey, 2014a).

In 2001, NGS increased the availability and value of the CORS by creating the web-based utility Online Positioning User Service (OPUS). This service allowed individuals to submit dual frequency GPS data and utilize algorithms that processed their data into positions relative to the CORS. A result, utilizing the rapid ephemeris for adjustments would even be available in a few minutes from submission (Stone, 2006).

OPUS is essentially a post-processing technique of averaging single-baseline solutions from the collected RINEX file data to three nearby CORS stations who collected data simultaneously. Specifically, OPUS utilizes Program for the Adjustment of GPS Ephemerides (PAGES) software utilizing double difference, carrier-phase measurements to compute ITRF referenced baseline vectors (Stone, 2006). The selection of the CORs stations to be used can be either manually entered or automatically chosen by OPUS. If OPUS is allowed to choose, an iterative process investigating the quality of the solution is conducted beginning with the closest CORS. If the CORS are deemed not to fit pre-specified standards for quality or quantity, then it will be rejected and the search moved outward until three CORS are eventually selected (Stone, 2006). Solutions are then returned to the user via emails containing coordinates of the mark with either a standard deviation solution for sessions less than 2 hours or a peak to peak error solution for solutions greater than 2 hours. OPUS only processes the GPS data utilizing 30 second epoch rates, and any data captured in smaller increments will not affect its solution (Stone, 2006).

OPUS is broken into two major categories that include OPUS-Static and OPUS-Rapid Static. OPUS-Static (OPUS-S) consists of data that must be between 2 hours and 48 hours in length (Schenewerk et al., 2012). OPUS Rapid-Static (OPUS-RS) processes observation data spanning from 15 minutes until 2 hours. Data from either time set are processed slightly differently due to different algorithms used to resolve unique errors of shorter sessions and a greater difficulty in modeling weather delays.

Although, OPUS-RS is more convenient in time collection requirements, its data lacks a significant degree of accuracy in relationship to OPUS-S. This problem of decreased accuracy is not solely an OPUS-RS issue with many other processing packages resulting in large errors due to little or no change in satellite geometry and similar atmospheric conditions. These problems reduce the ability to fix integers properly (Foote et al., 2006). For instance, NGS posts an OPUS-RS Map which presents predicted relative precisions of solutions at any given point. The map overlay is updated weekly using the most recent CORS data and displays either a 1 sigma error solution for either a 15 minute or 60 minute observation. For this study's area, the predicted OPUS-RS solution at 95% confidence for 15 minutes ranges from 5.6-7.2 cm and for 60 minutes was 3.4 – 4 cm of precision (National Geodetic Survey, 2014b). This precision level is not sufficient to achieve the goal of 2 cm accuracy at 95%. Another study in 2008 that focused on 15 minute data spans at the same time of the day during the tenth day of ten consecutive months resulted in a vertical standard error of 3-4 cm for heights in Oregon (Schwarz et al., 2009).

Accuracy increases with the utilization of OPUS-S. For instance, one study utilizing 2 hours of data shows resulting RMS errors in northing, easting, and up of 0.8, 2.1, and 3.4 cm respectively (Foote et al. 2006). A different study utilizing

861 data files collected at 227 stations in central Texas, during the summer of 2011 showed that OPUS-S solutions show the mean RMS double difference phase residuals dropped to about 3.9 mm (Schenewerk et al., 2012). Even though these accuracy levels are an improvement, they still do not achieve 2 cm accuracy with only 2 hours of data.

NGS recommends several ways to improve the accuracy of OPUS. First, NGS recommends that one should observe longer to fix additional ambiguities and reduce multipath error. Second, NGS suggests that one should observe again and ideally with a different observer, different equipment, on a different day, and at different time of the day. This is to ensure that similar errors are not reproduced resulting in a solution closer to the truth. Next, NGS states that one should wait a day to submit to OPUS allowing for a more accurate ephemeris and more refined CORS positioning data. Lastly, NGS recommends processing the data oneself by utilizing software that includes outlier detection, change in tropospheric parameters, change to mask angles, and network configurations (Ugur, 2013). In fact, NGS has created such a software to assist called OPUS-Projects (OP).

OP is an online processing package maintained by NGS and free to the public. It utilizes OPUS tools for uploading, processing, and sharing geodetic network solutions and baseline processing of simultaneous GPS observations solutions, and it allows the user to create sessions, baselines, and conduct a least squares adjustment on one or more sessions (Armstrong, 2014). In order to fully utilize OP, one needs to attend a training session conducted by NGS to gain a mangers account. One of the central concepts behind OP is that one or multiple managers can create multiple or unique GPS projects shareable with field crews (Schenewerk et al., 2012). At least two hours of continuous data is entered into OP via the OPUS website, and the user must indicate what project the RINEX file and the resulting OPUS solution will be sent to. The manager can then determine which stations to constrain, the level of constraint, the tropospheric model, and some general network design aspects for each session prior to conducting an overall network adjustment. OP then combines the sessions using GPSCOM to produce a single output in the form of a SINEX file. OP then reports the final set of coordinates for each reference station and observed station (Weston et al., 2007).

5.2 USING OP

The next step in analyzing this study's data was to utilize OP and compare the results to the previous adjustment results from STAR*NET. First, all of the original RINEX data was uploaded into the OP project via the OPUS website. 24 hour RINEX files for every day of observation were also loaded into OP covering station LCS1 which is a station in the Oregon Real-Time GPS Network (ORGN). This station is located in Albany, Oregon, and the station was relatively close to all of the marks in the study. This station was used as a hub in the network and the details of this will be further explained later. Since OPUS supports the zipping and batching of data, this process of uploading was very fast. After all of the data was uploaded to OP, several steps were needed to process the data in OP. These steps are outlined below with greater detail on certain aspects. The results will then be presented in comparison to the traditionally processed NGS 58 and 59 STAR*NET data.

Regrettably, the initial planning and execution of this study did not fully consider processing the data within OP. One mark failed to have a full 2 hour session and not all marks had repeat observations of 2 hours or greater which would have made the OP solution more robust. For future studies, taking in the limitations of OP should be considered when planning session times. However, the power of OP processing can still be compared using the data available.

After the data was uploaded into the project, OP automatically divides the data into sessions according to the manager's pre-set criteria for the time overlap required for a single session. Marks that do not fit the manager's initial parameters for: minimum % of observations used, minimum % ambiguities fixed, maximum solution RMS, maximum height uncertainty, and maximum latitude and longitude uncertainties will be flagged for awareness, but may still be used in processing. The next step is to process each session individually. This step requires the user to identify which CORS to use as control and to create a network design for that session. This network design is perhaps the most critical aspect, because it is what will be used as the baseline connections for the final adjustment. The results of these session solutions can be analyzed individually or in comparison to the OPUS-Static confidence intervals, and the resulting coordinates from the session solutions will be used as the a priori coordinates for the final least squares adjustment. The last step is to add sessions into an overall network adjustment. After a complete adjustment is performed, the user can view any individual mark and compare their OPUS-Static solutions, different session solutions, and the overall network solution in order to detect any one day or session that may be an outlier. The network's solution contains coordinates for each mark along with final standard deviations and specific baseline data such as baseline length, RMS, % observations omitted, % fixed, and the list of sessions that contain each respective baseline.

The OP processing for this study also followed a similar workflow as before in regards to processing the network for northing, easting, and ellipsoid heights, followed by orthometric processing using valid NAVD88 benchmarks as constraints. Many of the user preferences were tested for optimum processing results and will be discussed below with recommendations on how to produce the most accurate results for small projects of similar size to this study's network. The results of this processing will also be presented below along with recommendations for improvement to OP.

5.3 OP Network Design

Prior to planning each session, users of OP must first decide on a network layout. OP lacks the ability to select every baseline a user may want. Instead it uses computer algorithms to connect baselines. The baselines that connect stations in any one session can be selected from one of the three types of network designs created by OP or a fourth user designed network. However, even the user designed network must still follow algorithm rules set by OP. The specifics of these types of networks will be discussed below. A general programming consistency between all of the OP network designs is that they must connect to a hub or series of hubs. NGS uses the term “hub” to signify, “a mark that is preferentially selected for inclusion in baselines” (Armstrong, 2014).

5.3.1 OP Default Network Designs

OP contains three different default network designs which include: nearest CORS, Minimum Spanning Tree (MST), and Triangle Network Design (TRI). The CORS design makes all CORS sites hubs, and each surveyed mark is connected to the nearest hub. The purpose of this design is to maximize on the CORS data. NGS reiterates that CORS should be carefully selected for quality, and this aspect will be further discussed later in this thesis under section 5.4.1.1.

The MST network design creates a minimal spanning tree design. This algorithm seeks the shortest possible baseline lengths between all marks. In essence, the program makes every mark a hub and connects them by the shortest baselines possible while maintaining only independent baselines. The theory behind using this design is to maximize the shortest baselines possible in order to reduce GPS errors. A problem in using this design can occur when no CORs or control points exist between a series of marks. This scenario could propagate error successively leading to poorer results (Armstrong, 2014).

The TRI network stands for Triangle Network Design, and this algorithm selects baselines by using Delaunay triangulation. In essence, this method connects marks so no point falls inside any other triangle while ignoring all other lines. TRI network design method also incorporates trivial vectors into the solution. Although these three are the default designs, the fourth option or USER design in OP allows the most flexibility for the individual user and was ultimately selected for this study and the comparison to STAR*NET.

5.3.2 USER CENTRAL HUB DESIGN

A USER defined network in OP is one that the user controls which marks are selected as hubs. Specifically, the user can toggle between all or some of the CORS or other project marks as hubs, and the user can select as many or as few hubs as desired. Therefore, somewhat different network patterns result compared to the three default configurations. A significant advantage of this network is the central hub design.

The central hub design is a network design that uses one hub or multiple central hubs if the network is spread over very large areas. This single hub must be within 100 km of the project area, but it should be as close as possible to all of the statically observed marks within the network. This central hub is then connected directly to multiple control stations or CORS, and the hub is also connected directly to all of the statically observed sites. Moreover, the HUB must be present in all sessions on all days. Following these rules results in remarkably precise and accurate results if the HUB is also a mark that is observed for 24 hours during the static session days. Therefore, the recommended practice is to allow a 24 hour active station act as a hub and allowing all of the other CORS or control to determine its position. The baselines from the hub then allow the other static marks to be determined. An example of a central hub design is depicted in Figure 33 below.

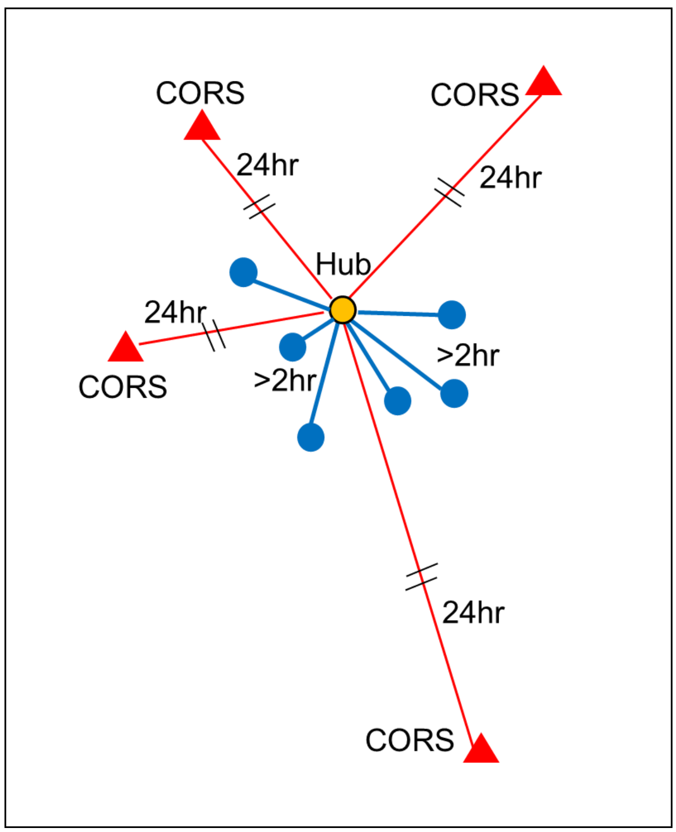

Figure 33: Central Hub Network Example

The central hub design has several benefits over other network designs. If 24 hours of data are used and the hub is allowed to float, the control stations or CORS will essential fix its position based on their combined 24 hour observations. If multiple days of a survey campaign were collected, then the hub's position will be based on all of the data. This resolves in the hub's position at a very tight precision and accuracy. The rest of the network is then adjusted from the hub's determined position. As a result, the position of the static marks are not as effected by errors at other marks. This theory works best when the hub has matching 24 hours of data with the control. Because of the resulting increases in accuracy and precision, this network design was the one used for this study. Next, the other parameters for OP processing needed to be determined.

5.4 OP Parameters Testing

OP has very few parameters that the user can select from, but the testing conducted for this study suggests that these parameters can effect results. First, the control stations and their level of constraint must be selected. Also, the tropospheric model and the tropospheric data collection intervals must also be determined. Research and testing on which parameters were ideal was conducted and will be explained in the following sections.

5.4.1 Determining Control

OP is designed primarily to use CORS stations as control for all networks processed in the program. However, other sites such as the International GNSS Service (IGS) can also be used as control. The key in determining which control marks to use is not an exact science, but several guidelines should be followed. First, all control stations should be manually chosen. A user could allow OPUS to automatically choose CORS for the uploading of data into OP via OPUS-Static, but this may result in less accurate control. Research should be done by any user to determine which control points have a suitable degree of precision and accuracy based on the mean and standard deviations of their data over a period of days, weeks, or even months. These details along with advantages and disadvantages of using the CORS network or the IGS network will now be discussed.

5.4.1.1 Continuously Operating Reference Station

Control in OP starts with the CORS network. Any CORS station can be added to OP even CORS stations that are no longer active but may have been during the time of a survey. First, the CORS that were initially selected while uploading the RINEX files to OPUS-Static are automatically included in OP along with two other nearby CORS. Any other CORS must be manually loaded into the project. The CORS available are listed in OP as: already in project, existed > 7 years (recommended), existed < 7 years (use freely), existed < 5 years (use with caution), existed < 3 years (avoid if possible). The rationale behind these time standards pertains to the velocities of the marks and the ability of NGS to model them throughout time. OP also allows the user to deselect during processing any CORS that was previously loaded.

Research on the exact repeatability and relative accuracy for any potential CORS should be conducted by the user prior to selection. First, local “experts” such as members of a state’s Department of Transportation or NGS employees who have processed networks with multiple CORS in the area should be consulted, and for this study, Oregon’s NGS advisor Mark Armstrong was consulted on which CORS were traditionally good. Second, research can be conducted by analyzing the CORS 90 day time series. This data is available online via NGS site specific CORS pages available at URL: [geodesy.noaa.gov/CORS/] (National Geodetic Survey, 2014a). The 90 day or (short-term) time series plot depicts the mean northing, easting, and up for a particular station over the past 90 days and also displays the standard deviations of those values. An example 90 day time series obtained from the NGS website is depicted in Figure 34 below.

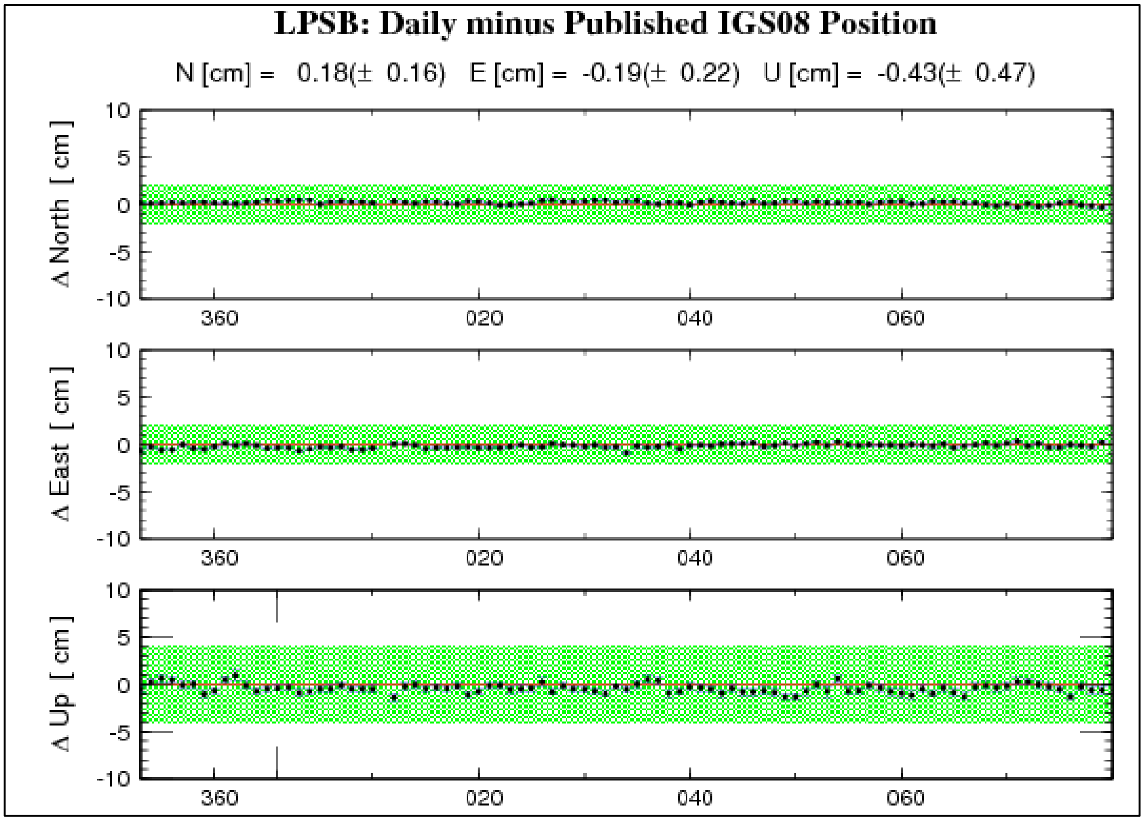

Figure 34: Example Short-Term Time Series from CORS Website

These short-term time series allow the user to detect a constant bias for a particular station and to see if the station is relatively repeatable. Ideally, these plots should be saved for all potential CORS with the actual survey days in the very middle of the 90 days. This would allow for the best selection of CORS and ensure there were no anomalies during the actual survey sessions. Regrettably, this fact was not discovered for this study until very long after the survey data was collected, but several CORS were selected based on the current 90 day time series and the reputation of the local CORS from the state NGS advisor.

Arguably the biggest advantage of using CORS as a control is that this can allow the user to select the closet and most accurate CORS in order to maintain as short of baselines as possible. Traditionally this concept was considered ideal. However, research is showing that this may not be the case due to unresolved errors within the troposphere. Therefore, another network for control such as the IGS was also considered.

5.4.1.2 International GNSS Service

The IGS is an international organization made up of hundreds of agencies, universities, and research institutions throughout the world that work together to produce the highest precision GPS satellite orbits in the world. This organization maintains a network of over 400 continuous reference stations, and are generally more regulated than the CORs network. For purposes of this paper, the acronym IGS will be used to refer to this network of continuous reference stations and their published coordinates (International GNSS Service, 2015).

IGS has several benefits compared to CORS when utilized as control within OP. First, studies analyzing OPUS and OP show that for static GPS processing little relationship exists between coordinate accuracy and GPS vector length as long as a sufficient number of satellites are dually observed to fix integer ambiguities and to correctly estimate the tropospheric delay (Eckl et al., 2001). Moreover, the CORS network and all OPUS tools operate and process vectors within the latest reference frame of the IGS (Smith et al., 2014). Therefore, utilizing IGS directly as control stations reduces the need to convert coordinates from a different reference system into the IGS even though this conversion may only result in sub-millimeter error.

A primary reason for selecting IGS as control is more accurate resolution of the central hub's location due to more accurate reduction of tropospheric delay. One study conducted by the Ohio State University and NGS shows that the determination of tropospheric corrections and ellipsoid heights are highly dependent on baseline length, reference stations used, network configuration, and session duration. Furthermore, the analysis confirmed that long baselines greater than 300 km provided a more realistic tropospheric corrections than shorter baselines. The recommendations of this study also stated that in order to reduce possible errors associated with reference station coordinates, IGS reference stations should be included within the recommended multiple reference station configuration in order to stabilize the network (Mader et al., 2013). One of the co-authors of this study Dr. Grejner-Brzezinska also committed on this same issue in a report to the Ohio Department of Transportation and specifically stated that the:

> multiple base approach (combination of CORS and IGS stations) is the optimal network, which improved the estimation of the tropospheric corrections, the quality of the processing results, and the positioning accuracy, especially in the height component. This configuration would reduce the possible errors associated with the base station, provide reliable tropospheric corrections and improve the accuracy of the ellipsoidal heights. These test cases also illustrated that a longer session provides higher accuracy and reliable

> ellipsoidal heights. Based on the results in this study at least a two-hour data span should be used to determine the ellipsoidal heights accurately in OPUS-Projects. Additionally, a second independent observation should be used to increase the confidence in the processing results. In order to maximize independence of the observations, the second observation should be obtained on a different day and at a different time of day (Grejner-Brzezinska and Toth, 2013, pp 56-57).

These studies support the utility of utilizing long baselines with IGS stations which was also tested in this project.

Another reason to test the IGS as reference control stations is that this method is recommended in Section 3 of the OP User Instructions and Technical Guide. Specifically, the manual mentions the benefits of IGS stations and states:

> For large and/or important projects, include IGS (International GNSS Service) stations (sites) as part of the global network control. This:
> - Provides for an alignment of your survey to an accurate global reference frame.
> - Improves troposphere determination and resulting heights (Armstrong, 2014, pg 78).

Also, the manual makes reference to the central hub design and utilizing IGS stations by stating:

> In Figure 3.3, a single CORS (hub) with 24 hours of data is connected to several IGS stations (outside the mapped region) so the network is strongly connected to the global reference frame. The IGS sites are tightly constrained while the hub is not constrained (loose), and is free to be positioned by the IGS sites within the adjustment. The other marks are also adjusted relative to the IGS sites because of their connecting baselines to the hub. This provides a consistent reference frame for marks and is suitable for projects up to several hundred kilometers with between 2 and 4 hours of data (Armstrong, 2014, pg 80).

Due to these recommendations, both a mixture of CORs alone, CORs and IGS, and IGS alone were tested in OP to decide which control configuration is best suited for processing the network.

5.4.1.3 CORS vs. IGS

In order to test the differences of using CORS vs. IGS, the results of the different control configurations within OP were compared to the published values of marks and the results were compared to the earlier derived NGS 58 and 59 results using LGO and STAR*NET. This process allowed trends to be identified, and eventually recommended practices for control configuration was determined. First, some initial decisions were made on which CORS and which IGS stations to use.

For the CORS station, a list of consistent CORS with a good reputation were obtained from Oregon State NGS advisor Mark Armstrong. The short-term time series for all possible CORS were also analyzed. The general criteria used for selection was that all of the CORS had to be within 250 km of the project site and the mean northing, easting, and up had to be less than 0.4cm from the published coordinates. The standard deviations of the short-term time series also all had to be less than 0.5 cm. This criteria ensured the CORS were relatively consistent and accurate. Lastly, the CORS were selected in a manner to ensure the geometry of the baselines surrounded the hub across a 360 degree span as much as possible. Baselines from multiple angles helped to reduce error shifting the hub in any one given direction by providing corrections from opposing directions. After selecting potential candidates for control, the OP network was processed utilizing a minimally constrained adjustment in order to see if any one CORS moved significantly from its published coordinate. In the end, nine different CORS were used as control around the project site which include stations: GOBS, REDM, LFLO, RSBG, LPSB, P367, DDSN, P420, and P698. A diagram depicting the control layout is depicted in Figure 35 below and is an actual screenshot from the OP program.

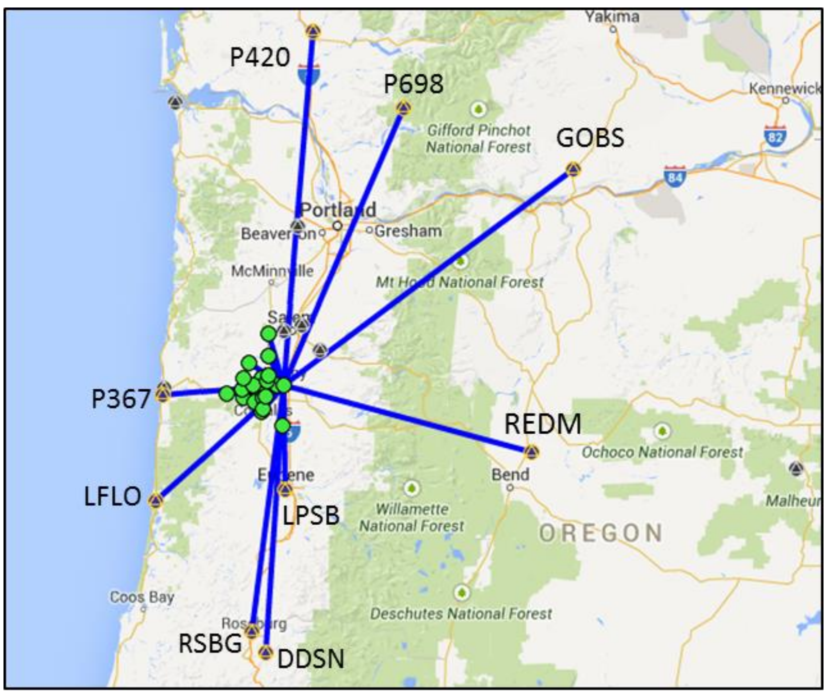

Figure 35: OP CORS Only Network

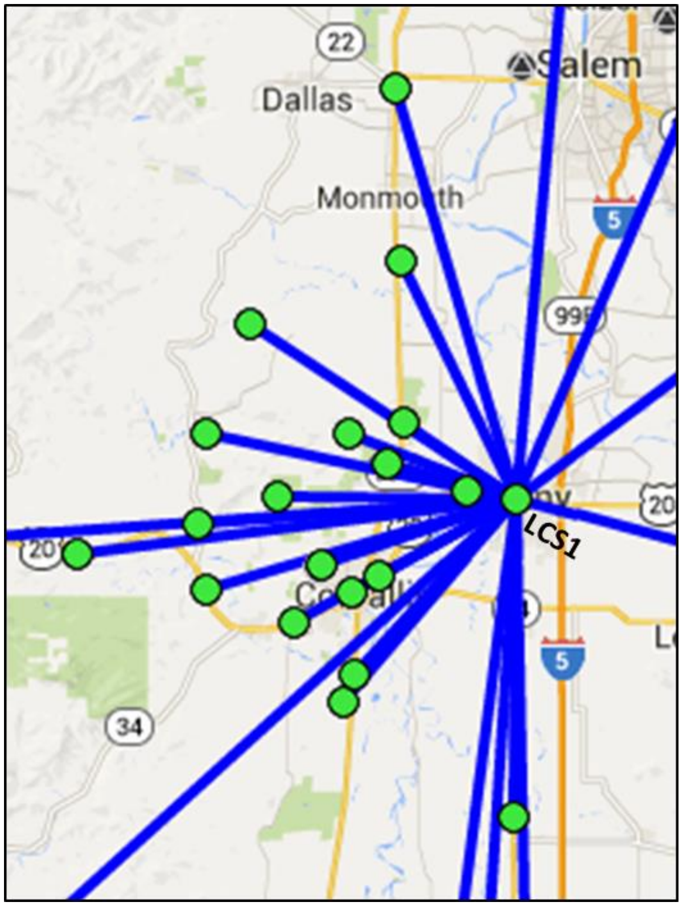

Figure 36: Blowup of CORS Only Network

For selecting IGS stations to be tested, the same geometry criteria of surrounding the project area as used for the CORS was utilized. However, IGS stations are not as prevalent as the CORS. Repeatability was checked via the IGS website in a similar fashion to the CORS. After researching different stations, six different IGS stations were selected for testing in OP which include stations: AMC2, CHWK, HOLB, JPLM, PRDS, and QUIN. An OP screenshot showing the control layout is presented in Figure 37 below.

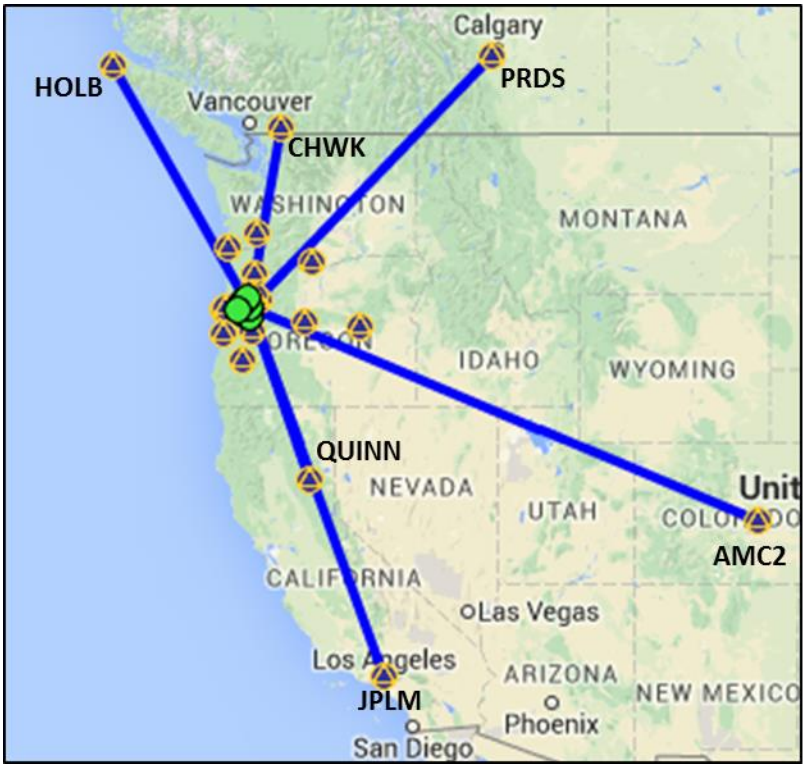

Figure 37: OP IGS Only Network

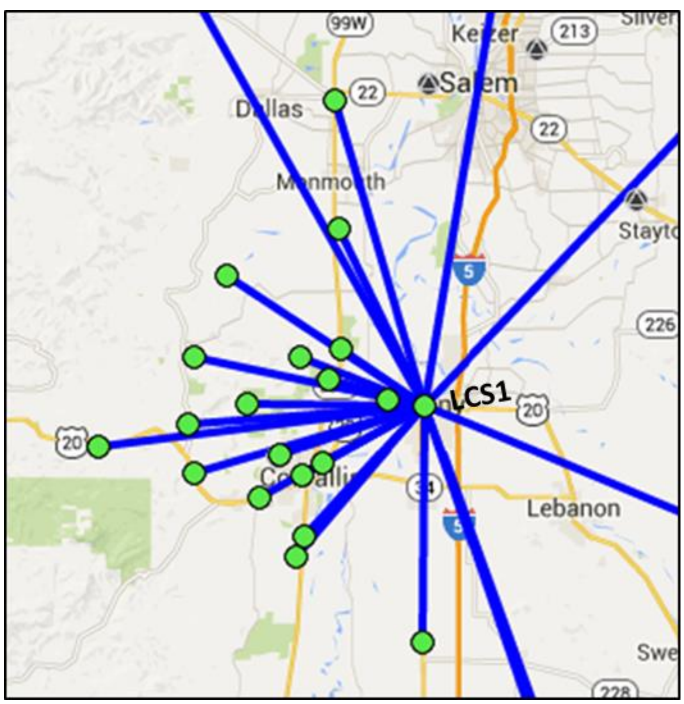

Figure 38: Blowup IGS Only Network

First, CORs alone as the control configuration was tested followed by a combination of CORs and IGS. Lastly, IGS alone was tested. Examples of the CORS only and the IGS only networks are depicted in figures IGS only and CORs only above, and an OP screenshot figure portraying the combination of IGS and CORS is in Figure 39 below.

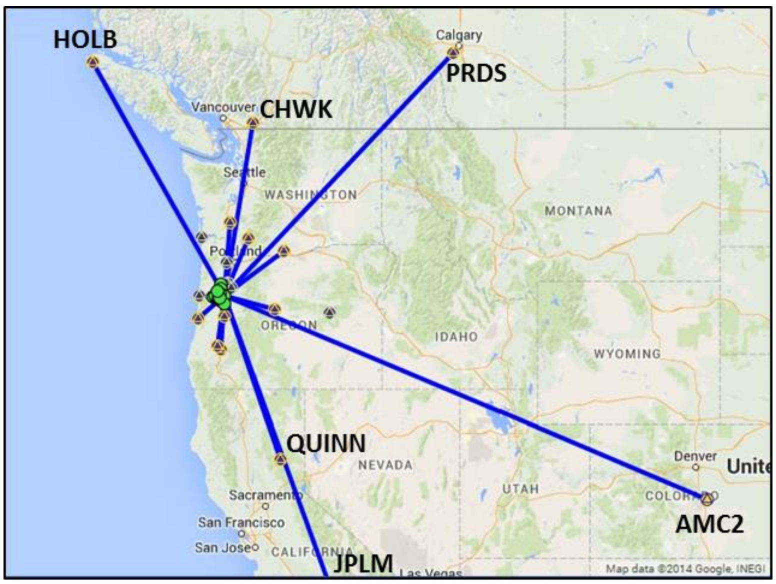

Figure 39: OP IGS and CORS Network

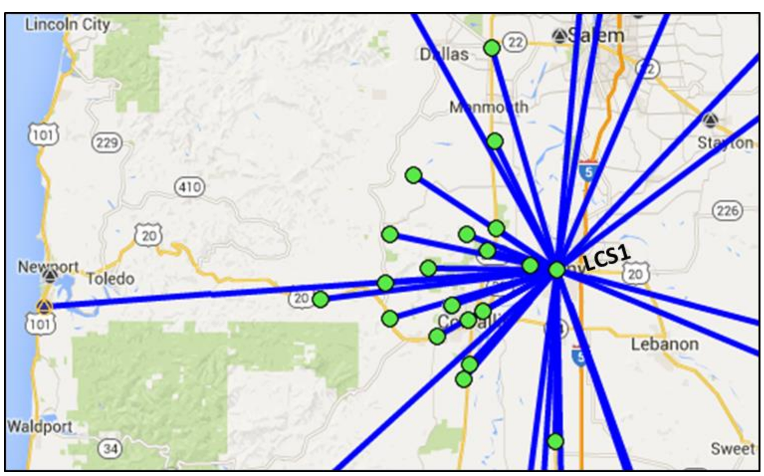

Figure 40: Blowup of IGS and CORS Network

The results of these tests are present in Tables 12-14. A clear trend on the relative accuracy and precision compared to the published and earlier derived coordinates was discovered. For instance, when only CORS was processed with a tight constraint, the average difference in ellipsoid heights from published was 2.0 cm and the average difference from the earlier conventional network processed in STAR*NET was 8 mm with a max difference of 4.0 cm and 3.1 cm respectively. When processed with a combination of IGS and CORS, the comparisons dropped to an average of 1.9 cm difference from published and 7 mm of difference with the STAR*NET results. The max difference also dropped to 3.8 cm and 3.1 cm. Lastly, when IGS alone was utilized as control the network dropped to an average difference of 1.5 cm from published and 4 mm from the earlier NGS 58 results. The maximum range also dropped to 3.5 cm and 3.0 cm of difference. The same general trends for an increase in precision and accuracy also occurred in the northing and easting coordinates and all three results can be viewed in Tables 12-14 below.

Table 12: Control Influences on Ellipsoid Height (meters)

	CORS only TIGHT		IGS/CORS TIGHT		IGS only TIGHT	
Station	diff Published (m)	diff STAR*NET (m)	diff Published (m)	diff STAR*NET (m)	diff Published (m)	diff STAR*NET (m)
U727	***0.032***	0.006	***0.032***	0.006	***0.025***	-0.001
G728	***0.022***	0.014	***0.021***	0.013	0.015	0.007
NESMITH	0.013	0.018	0.011	0.016	0.007	0.012
BICKFORD	0.017	0.017	0.017	0.017	0.013	0.013
S714	0.019	-0.006	0.019	-0.006	0.016	-0.009
N99RESET	0.012	***0.026***	0.012	***0.026***	0.008	***0.022***
J99	***0.024***	***0.031***	***0.024***	***0.031***	***0.023***	***0.030***
Y683	0.010	0.005	0.011	0.006	0.010	0.005
BEEF	***0.040***	0.013	***0.038***	0.011	***0.035***	0.008
PRICE	***0.022***	0.014	***0.023***	0.015	0.019	0.011
G287	***0.024***	0.009	***0.023***	0.008	0.018	0.003
J54	0.001	0.002	-0.002	-0.001	-0.006	-0.005
T714	***0.031***	***0.024***	***0.030***	***0.023***	***0.026***	0.019
CORVA	0.008	0.001	0.007	0.000	0.002	-0.005
PEAV	N/A	-0.010	N/A	-0.012	N/A	-0.012
MAG	N/A	0.017	N/A	0.014	N/A	0.013
Z714	N/A	-0.012	N/A	-0.011	N/A	-0.010
Q388RESET	N/A	0.017	N/A	0.013	N/A	0.010
PEAK	N/A	***-0.027***	N/A	***-0.031***	N/A	***-0.036***
Average	0.020	0.008	0.019	0.007	0.015	0.004
Max	0.040	0.031	0.038	0.031	0.035	0.030
Min	0.001	-0.027	-0.002	-0.031	-0.006	-0.036
# > 2cm	7	4	7	4	4	3

Table 13: Control Influence on Northing (meters)

Station	CORS only TIGHT diff Published (m)	CORS only TIGHT diff STAR*NET (m)	IGS/CORS TIGHT diff Published (m)	IGS/CORS TIGHT diff STAR*NET (m)	IGS only TIGHT diff Published (m)	IGS only TIGHT diff STAR*NET (m)
U727	0.003	-0.008	0.004	-0.007	0.006	-0.005
G728	-0.001	-0.002	0.001	0.000	0.002	0.001
NESMITH	-0.008	-0.013	-0.007	-0.012	-0.004	-0.009
BICKFORD	-0.005	-0.005	-0.003	-0.003	-0.002	-0.002
S714	-0.009	-0.008	-0.009	-0.008	-0.006	-0.004
N99RESET	-0.003	-0.013	-0.001	-0.011	0.001	-0.009
J99	0.000	-0.009	0.001	-0.008	0.004	-0.005
Y683	0.002	-0.010	0.004	-0.008	0.006	-0.006
BEEF	-0.002	-0.006	0.000	-0.004	0.002	-0.002
PRICE	***0.017***	-0.011	***0.017***	-0.011	***0.018***	-0.010
G287	0.003	-0.007	0.004	-0.006	0.005	-0.005
J54	***-0.042***	0.002	***-0.042***	0.002	***-0.042***	0.002
T714	0.008	-0.001	0.011	0.002	0.012	0.003
CORVA	0.007	-0.002	0.008	-0.001	0.010	0.001
PEAV	-0.011	-0.011	-0.007	-0.007	-0.010	-0.010
MAG	-0.003	-0.003	0.000	0.000	0.001	0.001
Z714	-0.001	-0.001	-0.001	-0.001	0.000	0.000
Q388RESET	0.006	0.006	0.006	0.006	0.006	0.006
PEAK	-0.002	-0.002	-0.001	-0.001	-0.009	-0.009
Average	-0.002	-0.005	-0.001	-0.004	0.000	-0.003
Max	0.017	0.006	0.017	0.006	0.018	0.006
Min	-0.042	-0.013	-0.042	-0.012	-0.042	-0.010

Table 14: Control Influence on Easting (meters)

Station	CORS only TIGHT		IGS/CORS TIGHT		IGS only TIGHT	
	diff Published (m)	diff STAR*NET (m)	diff Published (m)	diff STAR*NET (m)	diff Published (m)	diff STAR*NET (m)
U727	0.008	0.006	0.006	0.004	0.004	0.002
G728	0.009	0.009	0.008	0.008	0.006	0.006
NESMITH	0.003	-0.004	0.002	-0.004	0.000	-0.006
BICKFORD	0.009	0.009	0.008	0.008	0.006	0.006
S714	0.011	0.007	0.010	0.006	0.008	0.004
N99RESET	0.011	0.001	0.009	-0.001	0.008	-0.002
J99	-0.007	0.001	-0.009	-0.001	-0.011	-0.003
Y683	0.006	0.010	0.005	0.009	0.003	0.007
BEEF	0.003	0.002	0.001	0.000	0.002	0.001
PRICE	***0.019***	-0.001	***0.018***	-0.002	***0.017***	-0.003
G287	0.010	0.003	0.010	0.003	0.007	0.000
J54	***-0.052***	0.007	***-0.052***	0.007	***-0.052***	0.007
T714	0.003	0.003	0.003	0.003	0.004	0.004
CORVA	0.010	0.015	0.008	0.013	0.005	0.010
PEAV	0.004	0.004	0.002	0.002	0.002	0.002
MAG	0.005	0.005	0.005	0.005	0.004	0.004
Z714	0.003	0.003	0.003	0.003	0.002	0.002
Q388RESET	0.009	0.009	0.010	0.010	0.010	0.010
PEAK	0.006	0.006	0.002	0.002	-0.005	-0.005
Average	0.004	0.005	0.003	0.004	0.001	0.002
Max	0.019	0.015	0.018	0.013	0.017	0.010
Min	-0.052	-0.004	-0.052	-0.004	-0.052	-0.006

The trend suggests that the IGS stations are the superior stations to utilize for control. The reasons for this increase in accuracy is most likely due to the fact that IGS stations are more heavily monitored and regulated than CORS sites. Additionally, the tropospheric delay is more accurately resolved with IGS alone. The values of the zenith delay obtained in the OP results were also reduced with the longer baselines to the central hub from IGS. The shorter baselines from the central hub to the static marks resolved the local GPS errors that are conventionally resolved with short baselines. Therefore, IGS alone combined with a 24 central hub site was determined as the recommended practice for control. This configuration may not be suited for other survey networks, but the central hub design combines the benefits of the long baselines from the IGS and the shorter baselines within the project.

5.4.2 Levels of Constraint

OP also allows different levels of constraint for the control sites which include loose, normal, and tight. The OP manual states that loose allows up to one meter of float for the constrained points while normal allows up to one centimeter of float. The manual also states that tight allows up to one tenth of a millimeter (Armstrong, 2015). These statements along with the ideal constraint for the central hub design were tested.

In order to test the levels of constraint, the same methodology previously conducted for the CORS vs. IGS section was used and the resulting coordinates derived in OP were compared to the published and earlier STAR*NET results. The loose setting was not tested because allowing a control point to float one meter was considered far too loose for obtaining accurate results. Both the normal and the tight settings were tested using a CORS only and later an IGS only configuration.

Conventional wisdom suggests that normal constraints (1 cm standard deviation for control) should be used in order to allow the control coordinates to move slightly, but the OP results do not support this guideline. The results show that the control coordinates moved more than reasonable under normal constraints and in effect allowed the free marks to shift the network. These movements result in a change in the central hub station LCS1 that in turn heavily effects the final derived coordinates to the static marks. Table 15 below highlights the changes between normal and a tight constraint.

Table 15: OP Constraint Influence on Ellipsoid Heights

Station	CORS only NORMAL		CORS only TIGHT	
	diff Published (m)	diff STAR*NET (m)	diff Published (m)	diff STAR*NET (m)
U727	***0.036***	0.010	***0.032***	0.006
G728	***0.025***	0.017	***0.022***	0.014
NESMITH	0.015	***0.020***	0.013	0.018
BICKFORD	***0.020***	***0.020***	0.017	0.017
S714	***0.022***	-0.003	0.019	-0.006
N99RESET	0.015	***0.029***	0.012	***0.026***
J99	***0.028***	***0.035***	***0.024***	***0.031***
Y683	0.013	0.008	0.010	0.005
BEEF	***0.042***	0.015	***0.040***	0.013
PRICE	***0.025***	0.017	***0.022***	0.014
G287	***0.025***	0.010	***0.024***	0.009
J54	0.004	0.005	0.001	0.002
T714	***0.034***	***0.027***	***0.031***	***0.024***
CORVA	0.011	0.004	0.008	0.001
PEAV	N/A	-0.007	N/A	-0.010
MAG	N/A	0.018	N/A	0.017
Z714	N/A	-0.008	N/A	-0.012
Q388RESET	N/A	0.019	N/A	0.017
PEAK	N/A	***-0.024***	N/A	***-0.027***
Average	0.023	0.011	0.020	0.008
Max	0.042	0.035	0.040	0.031
Min	0.004	-0.003	0.001	-0.027
# > 2cm	9	6	7	4

Clearly, the control points seem to move too much when using the normal constraint setting on OP. This level of constraint in essence underweights the control points and allows the floating marks and their errors to propagate and influence the network. The tight constraint effectively fixes the central hub in position. This fact along with the multiple days of observation of the central hub results in a more accurate position of the hub. The errors at individual stations within the static observations also had less influence on the entire network. This in turn, results in the most precise coordinates for the static marks. Therefore, the conclusion was made that the recommended practice when using OP was to only use the tight constraint for all control marks.

5.4.3 Tropospheric Modeling

A proper tropospheric model helps to reduce atmospheric errors. This reduction along with the combination of short and long baselines while using the 24 hour central hub design helps to increase final accuracy. Short baselines essentially have the same satellite geometry at similar elevations and make it difficult to fully resolve tropospheric corrections. This could lead to an error in accuracy of heights that cannot be fully corrected for. Baselines at very large distances will see the same satellites if the observations are long enough in duration. These observations will also be from a significantly different geometry, and this will help to resolve tropospheric corrections more accurately leading to more accurate heights. In essence, this is why the IGS stations helped to resolve the central hub's position. However, the theory behind long baseline processing still requires a model to estimate the hydrostatic or dry component of the troposphere.

OP has two options for tropospheric models. First, the step offset model fits discrete corrections at set time intervals to model the delay according to time. On the other hand, the piecewise linear model fits a curved line across the average zenith delay according to time and moves more smoothly as opposed to the step offset model. Both models were tested and the results were compared. Ultimately, the piecewise linear with a sampling interval period of 2 hours was determined to be slightly more accurate than the step offset model, but the differences were minor. The slight improvement may be due to the fact that nature usually does not move in discrete steps, and the smoothed nature of the piecewise linear model is a better real world approximation.

5.5 OP PROCESSING

The general steps for following NGS 58 and 59 also applies to processing networks within OP. First, the network was processed via an ellipsoidal adjustment resulting in final coordinates for northing, easting, and ellipsoid heights. Next, the network was processed via an orthometric adjustment in order to determine elevations.

One of the advantages of OP over the conventional processing method is the ability to detect potentially bad data with little work. Since the RINEX data uploaded to OP is processed first through OPUS, large standard deviations or low fix rates will be flagged for the user’s attention. An example can be seen in the OP screenshot Table 16 below depicting all of the 2 hour marks uploaded from the study survey. OP presents the observations into sessions and labels them according to the GPS day. After final processing network solutions are also listed on this display.

Table 16: OP Session Table

MARKS	Sessions & Solutions 2014-195 A	2014-196 A	2014-196 B	2014-197 A	2014-197 B	2014-203 A	2014-205 A	2014-206 A	2014-210 A	2014-210 B	2014-211 A	2014-211 B	2014-212 A	2014-213 A	MARKS
beef						●		●							beef
bick	●	●			●		●								bick
crva											●		●	●	crva
g287										⊘	●		●	●	g287
g728	●	●			●										g728
j54d									●			●			j54d
j99d			⊘	⊘											j99d
lcs1	●	●	●	●	●	●	●	●	●	●	●	●	●	⊘	lcs1
magd						●	●	●	●	⊘		⊘			magd
n99d	●	●	●	●	●										n99d
nesd	●	●	●	●	●										nesd
peak														⊘	peak
peav						⊘		●							peav
pric										●	●		●		pric
q388									●			●			q388
s714	●	●	●	●	●	●									s714
t714						●		●							t714
u727	●	●			●										u727
y683			●	●											y683
z714									●	●	●	●			z714

After uploading through OPUS, OP automatically sorts the observations into common session periods and marks that are beyond the user's parameters for percent of observations used, percent ambiguities fixed, maximum solution RMS, maximum height uncertainty, and maximum latitude and longitude uncertainties are flagged ⦸ as can be seen in the table above at stations G287, J99, MAG, PEAK, and PEAV. OP also plots these marks on a map and any mark that contains at least one potential bad session is also flagged (Figure 41).

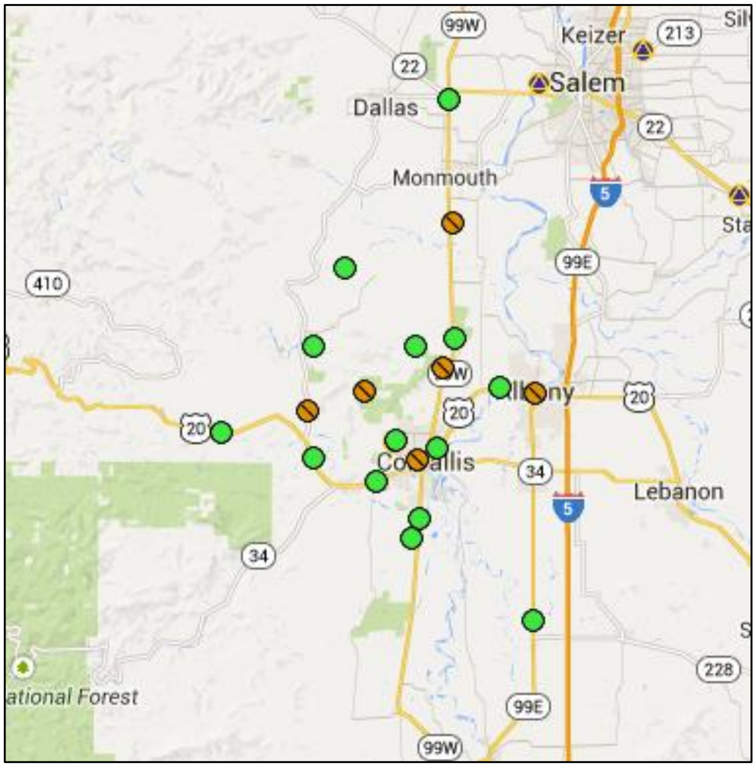

Figure 41: OP Initial Map

These initially bad sessions can still be processed, or the user can decide to leave their data out. Issues of multipath or other unforeseen errors not previously identified are also highlighted. For instance, the data at station J99 appears to portray signs of multipath due to large initial vertical OPUS errors of ±10.6 cm and ±6.8 cm at 95% confidence for the station's two different observations. For subsequent OP processing for this study, all of the potential bad data was removed as long as there was a good repeat observation on a particular station. However, if only bad data existed at a station, the data was still used in order to obtain a set of coordinates for that station. This was the case for stations PEAK and J99. Like J99, Station PEAK also had a large initial vertical error listed in OP at ±14.1 cm at 95% confidence. Normally, this type of data should be rejected, but it was left in order to test if OP can properly detect data that should be removed and to test the final accuracy and precision of these two points along with their effects on the entire network.

5.5.1 Ellipsoidal Adjustment

The process for the ellipsoidal adjustment in OP followed the same steps as specified in NGS 58. First, a minimally constrained adjustment holding only one IGS station fixed was conducted in order to check the consistency of the network. The adjusted coordinates versus the initial coordinates were compared for all of the control sites in order to ensure the relative accuracies of the control coordinates. After this step, the fully constrained adjustment was conducted for the ellipsoid and all six IGS stations were constrained with the tight parameters. The results were then obtained for northing, easting, and ellipsoid height. Specifics of the final ellipsoidal coordinates are available in Table 18 in the OP results section below.

5.5.2 Orthometric Adjustment

A similar procedure as specified in NGS 59 was also followed for the orthometric processing starting with a minimally constrained adjustment. First, one orthometric height was held fixed, and to match the method followed previously in STAR*NET, CORVA , a First Class II NAVD88 Benchmark, was the initial elevation fixed. Also, the horizontal 2-D coordinates for all of the IGS stations were held fixed which as a result of the least squares adjustment and the network design effectively holds the central hub's 2-D coordinates relatively fixed. After this first minimally constrained adjustment, all benchmarks within 2 cm of their published elevations were considered valid NAVD88 benchmarks. Results of the minimally constrained adjustment and the valid marks are depicted in Table 17 below. Interestingly, all but two benchmarks with published orthometric heights fell within the 2 cm range and were initially considered valid. Only G728 and J54 were outside of the 2 cm range.

Table 17: OP Valid NAVD88 Benchmarks

Station	**Published H_{88} (m)**	**OP H_{88} (m)**	**Difference (cm)**
U727	226.783	226.772	***-1.1***
G728	84.421	84.397	-2.4
NESMITH	63.923	63.917	***-0.6***
BICKFORD	75.072	75.078	***0.6***
S714	66.464	66.46	***-0.4***
N99RESET	75.430	75.419	***-1.1***
J99	76.442	76.437	***-0.5***
Y683	94.600	94.599	***-0.1***
G287	202.076	202.082	***0.6***
J54	86.285	86.253	-3.2
T714	69.481	69.49	***0.9***
CORVA	126.017	**Constrained**	**Constrained**
MAG	70.096	70.096	***0.0***
Z714	171.126	171.133	***0.7***

Once the valid NAVD88 benchmarks marks were determined, all of the valid marks were held fixed along with the horizontal coordinates for the IGS control stations and a fully constrained orthometric adjustment was conducted. The results were then checked for an over-constraint by comparing the minimally constrained and the fully constrained values and looking for differences between neighboring stations greater than 1 cm. One station, U727 had a change of 1.1 cm and its neighbors G287 and T714 had changes of -0.6 cm and -0.07 cm. Due to these facts, U727 was allowed to float and a final round of orthometric processing was conducted. The final results from the last round of orthometric processing are depicted in Table 18 in the OP results section below.

5.6 OP RESULTS

The OP ellipsoid adjustment results (Table 18) and the OP elevation results (Table 19) agree well with the published and the previous STAR*NET results. Specifics on these comparisons are shown below along with an explanation for outliers. Additionally, the error statistics reported by OP are critiqued.

Table 18: OP Ellipsoid Results

Station	OP h_{83} (m)	difference from published (cm)	difference from STAR*NET (cm)	OP Northing (m)	difference from published (cm)	difference from STAR*NET (cm)	OP Easting (m)	difference from published (cm)	difference from STAR*NET (cm)
U727	204.767	2.5	-0.1	107599.772	0.6	-0.5	2258120.923	0.4	0.2
G728	61.324	1.5	0.7	85650.516	0.2	0.1	2292155.689	0.6	0.6
NESMITH	41.526	0.7	1.2	143466.293	-0.4	-0.9	2284683.294	0.0	-0.6
BICKFORD	52.342	1.3	1.3	95243.441	-0.2	-0.2	2278864.623	0.6	0.6
S714	43.619	1.6	-0.9	111599.865	-0.6	-0.4	2289313.015	0.8	0.4
N99RESET	53.352	0.8	2.2	125462.709	0.1	-0.9	2272519.984	0.8	-0.2
J99	53.921	2.3	3.0	129935.728	0.4	-0.5	2284668.060	-1.1	-0.3
Y683	72.000	1.0	0.5	117245.757	0.6	-0.6	2284523.136	0.3	0.7
BEEF	120.526	3.5	0.8	116401.681	0.2	-0.2	2279988.254	0.2	0.1
PRICE	113.621	1.9	1.1	116815.540	1.8	-1.0	2268774.060	1.7	-0.3
G287	179.957	1.8	0.3	109659.145	0.5	-0.5	2267795.205	0.7	0.0
J54	63.695	-0.6	-0.5	101609.307	-4.2	0.2	2275282.556	-5.2	0.7
T714	46.778	2.6	1.9	105023.845	1.2	0.3	2282052.131	0.4	0.4
CORVA	103.473	0.2	-0.5	106082.956	1.0	0.1	2277508.140	0.5	1.0
PEAV	106.042	N/A	-1.2	113879.443	-1.0	-1.0	2283125.491	0.2	0.2
MAG	47.434	N/A	1.3	103730.645	N/A	0.1	2279875.967	N/A	0.4
Z714	148.851	N/A	-1.0	104413.264	N/A	0.0	2268368.639	N/A	0.2
Q388RESET	51.916	N/A	1.0	97228.418	N/A	0.6	2279769.813	N/A	1.0
PEAK	585.323	N/A	-3.6	111529.580	N/A	-0.9	2274265.345	N/A	-0.5

Overall, the OP ellipsoid height results compared closely with published. Over 70% of ellipsoid marks were within 2 cm of published ellipsoid heights and over 90% were within 3 cm with an average difference of 1.5 cm. When comparing the OP results to the previous results of NGS 58 and 59 derived in STAR*NET, 85% of the ellipsoid heights matched within 2 cm and 95% matched to 3 cm with an average difference of 4 mm.

For northings and eastings, the coordinates all matched the STAR*NET coordinates on average of 3mm and 2mm respectively with no difference between the two by more than 1.0 cm. The OP results also showed possible bad published northing and easting coordinates for both stations PRICE and J54. Station J54 is the mark that was earlier discussed as possibly being disturbed. The OP coordinates show station J54 differs by -4.2 cm and -5.2 cm in northing and easting from its published coordinates and those discrepancies match the STAR*NET solution within 2mm and 7mm, respectively.

Table 19: OP Orthometric Height Adjustment Results

Station	H_{88} (m)	difference published (cm)	difference STAR*NET (cm)
U727	226.773	-1.0	-1.0
G728	84.398	-2.3	0.1
NESMITH	63.923	Constrained	0.0
BICKFORD	75.072	Constrained	0.0
S714	66.464	Constrained	-1.3
N99RESET	75.430	Constrained	2.4
J99	76.442	Constrained	2.1
Y683	94.600	Constrained	0.0
BEEF	142.914	N/A	0.0
PRICE	135.646	N/A	0.1
G287	202.076	Constrained	-1.2
J54	86.253	-3.2	-1.4
T714	69.481	Constrained	0.0
CORVA	126.017	Constrained	0.0
PEAV	128.618	N/A	-2.1
MAG	70.096	Constrained	0.0
Z714	171.126	Constrained	-2.6
Q388RESET	74.661	N/A	0.2
PEAK	607.539	N/A	-4.5

For elevations, 90% of the OP solutions matched their published values within 2 cm and 95% matched within 2.5 cm. Only two marks in OP were found to be greater than 2 cm from their published elevations. When comparing to the STAR*NET results, 75% of the final elevations were within 2 cm of the STAR*NET results and 90% were within 2.5 cm of the STAR*NET results with an average difference of only 5 mm. One reason for the number of orthometric discrepancies between OP and STAR*NET was that OP found that stations N99-RESET, J99, and Z714 were all within 2 cm of published after conducting the minimally constrained adjustment and were deemed valid, but on the other hand, STAR*NET allowed these marks to float. The final differences for these three marks ranged from 2.6 cm to 2.1 cm.

Some outliers between the published values and the OP results were also identified. The maximum magnitude of differences for published ellipsoid heights was 3.5 cm at station BEEF which agrees within 8mm of the STAR*NET result previously presented in section 4.2.7. This mark most likely has an incorrect ellipsoid height published in the NGS IDB. Also the issues with J54 previously discussed in northing and easting also appears to be an issue in ellipsoid height as well according to OP. This mark was -3.2 cm off of the final ellipsoid height which compared within 1.4 cm of the STAR*NET results. The OP results also support that this mark was most likely disturbed since monumented and last surveyed by NGS.

Only two major outliers from the earlier NGS 58 and 59 results seem to exist. Specifically, J99 differed by 3.0 cm in ellipsoid heights and PEAK was off by -3.6 cm. PEAK had a -4.5 cm elevation difference between the OP and STAR*NET solutions. Other than these two marks, only Z714 was more than 2.5 cm at -2.6 cm in elevation between the OP and STAR*NET, and only N99 RESET had an ellipsoid height difference between OP and STAR*NET greater than 2 cm at 2.2 cm.

Several reasons may have caused the discrepancies from the published and the earlier derived STAR*NET values. First, PEAK was identified as having a problematic OPUS solution and only had one 2 hour observation with no repeat checks for redundancy. Station PEAK was also almost 300 meters above the second highest mark and was over 540 meters above the lowest mark in the network. Station J99 was also flagged as having bad data and may have had multipath effect its solution. Both of these stations were left in the OP processing in order to obtain processed coordinates with the foreknowledge that their results would be inaccurate. When these stations were processed in STAR*NET they had multiple baselines and multiple sessions that better resolved these possible errors.

OP presents the final coordinates with standard deviations of the coordinates. According to NGS programmer, Mark Schenewerk, the OP errors are taken from the formal, one standard deviation of the source solution. The solution is performed in the global reference frame (IGS08) as geocentric X, Y, and Z coordinates. Additionally, if coordinates can be transformed from the X, Y, Z to NAD83, then OP will present an orthometric height in the results and combine the geoid uncertainty to the error statistics (Schenewerk, 2015). For our results, the final reported errors listed in Table 20 below and includes both the ellipsoid and orthometric one sigma of error.

Table 20: OP Results Error Statistics

Station	Northing 1 sigma (cm)	Easting 1 sigma (cm)	Ellipsoid Height 1 sigma (cm)	Ortho Height 1 sigma (cm)
BEEF	0.0	0.0	0.2	1.5
BICFORD	0.0	0.0	0.1	C
CORVA	0.0	0.0	0.2	C
G287	0.0	0.0	0.2	C
G728	0.0	0.0	0.1	1.5
J54	0.0	0.0	0.2	1.5
J99	0.0	0.0	0.2	C
MAG	0.0	0.0	0.1	C
N99-RESET	0.0	0.0	0.1	C
NESMITH	0.0	0.0	0.1	C
PEAK	0.1	0.1	0.6	1.6
PEAV	0.0	0.0	0.2	1.5
PRICE	0.0	0.0	0.2	1.5
Q388-RESET	0.0	0.0	0.2	1.5
S714	0.0	0.0	0.1	C
T714	0.0	0.0	0.2	C
U727	0.0	0.0	0.1	1.5
Y683	0.0	0.0	0.2	C
Z714	0.0	0.0	0.1	C

These OP error statistics are very optimistic considering that the average standard deviation for all stations is only a tenth of a mm for northing and easting with only station PEAK having a different value than zero. The ellipsoid height statistics are also very precise with an average standard deviation of only 1.8 mm. The statistics for the orthometric heights are not as precise as our earlier work. OP reports the final elevations in terms of the national network accuracy and takes into account the error within the geoid model when determining the error statistics. The average standard deviation for the orthometric heights was 1.51 cm which is close to the geoid's error in the project area. Even with these tight error statistics, station PEAK seems to be an outlier in precision compared to the other stations.

5.7 RECOMMENDATIONS FOR IMPROVING OP

For an open source and web based software, OP processes networks to a similar level of accuracy of traditional commercial software. However, the largest weakness of OP is that it requires at least a 2 hour session to include in the network processing. If the shorter 30 minute sessions that NGS specified in their guidelines for NGS 58 and 59 could be included, the program would be more beneficial. These shorter sessions may produce solutions of varying accuracy, but other post-processing software packages can process short sessions and is an area that OP could improve on.

More user control and specific baselines is another recommendation for improvement. As currently programmed, OP only allows the MST, TRI, CORS, or USER network designs, and the ability to turn off or turn on specific baselines is not available. Other recommendations for improvement are minor and include: the ability to remove CORS previously loaded into a project, having the default error statistics reported at 95% confidence, allowing the addition of GLONASS observations, and the ability for the manager to create copies of the project. Lastly, OP network solutions for benchmarks should be shared with the surveying community on the OPUS-DB and not just the current policy of only being allowed to share 4 hour static solutions.

One area for future research within OP and in general is the manipulation of mask angles by session within OP. A correlation to changes in height occur when mask angles are altered. However, raising the mask level decreases the number of satellites seen by multiple stations, but it can possibly omit bad data from either multipath or lower angle signals that contain greater errors. An initial testing of changing the mask angle was conducted for this study, but no conclusive ideal angle was determined. Moreover, the ideal angle may not be the same for every session or every survey, but more accurate results may be possible when changing mask angles and this area deserves further research. For this study, only a 15 degree elevation cutoff was utilized for all methods of processing in order to ensure consistency.

6 RESULTS

6.1 FINAL COORDINATES (95% CONFIDENCE)

The tables below depicts the final derived coordinates at 95% confidence for all of the marks in the study. Both the STAR*NET and the OP solutions are contained. Table 21 shows the northing and eastings and latitude and longitude of each point. Table 22 shows both the ellipsoidal and orthometric height solutions, and the 95% confidence horizontal and vertical error ellipses are presented in Table 23.

Table 21: Final Horizontal Coordinates

Station	N#	OP N#	E#	OP E#	Latitude (° ' ")	Longitude (° ' ")
U727	107599.777	107599.772	2258120.921	2258120.923	44-35-38.08763	123-32-49.77083
G728	85650.515	85650.516	2292155.683	2292155.689	44-24-26.00398	123-06-35.23690
NESMITH	143466.302	143466.293	2284683.301	2284683.294	44-55-30.09921	123-13-40.97777
BICKFORD	95243.443	95243.441	2278864.617	2278864.623	44-29-22.26021	123-16-50.49572
S714	111599.870	111599.865	2289313.011	2289313.015	44-38-23.26498	123-09-22.19406
N99RESET	125462.718	125462.709	2272519.986	2272519.984	44-45-33.51447	123-22-26.11273
J99	129935.733	129935.728	2284668.063	2284668.060	44-48-11.97259	123-13-20.88347
Y683	117245.763	117245.757	2284523.129	2284523.136	44-41-20.91851	123-13-08.04470
BEEF	116401.683	116401.681	2279988.253	2279988.254	44-40-48.59266	123-16-32.56437
PRICE	116815.550	116815.540	2268774.063	2268774.060	44-40-49.19478	123-25-02.14067
G287	109659.150	109659.145	2267795.205	2267795.205	44-36-56.34623	123-25-34.79286
J54	101609.305	101609.307	2275282.549	2275282.556	44-32-44.34969	123-19-42.60465
T714	105023.842	105023.845	2282052.127	2282052.131	44-34-42.47173	123-14-41.34939
CORVA	106082.955	106082.956	2277508.130	2277508.140	44-35-11.71162	123-18-08.86717
D728	92359.210	N/A	2292487.798	N/A	44-28-03.58620	123-06-30.02184
PEAV	113879.453	113879.443	2283125.489	2283125.491	44-39-30.39064	123-14-06.30938
MAG	103730.644	103730.645	2279875.963	2279875.967	44-33-58.19246	123-16-17.92492
Z714	104413.264	104413.264	2268368.637	2268368.639	44-34-07.17234	123-25-00.21123
Q388RESET	97228.412	97228.418	2279769.803	2279769.813	44-30-27.53864	123-16-12.63118
PEAK	111529.589	111529.580	2274265.350	2274265.345	44-38-04.39265	123-20-44.48569

Northing/Easting: NAD1983(2011) Epoch 2010.00 SPC, Oregon North Zone, meters

Table 22: Final Vertical Coordinates

Station	Ellipsoid Height* (m)	OP Ellipsoid Height* (m)	Elevation^ (m)	OP Elevation^ (m)	Local Geoid Height (m)
U727	204.768	204.767	226.783	226.773	-22.02
G728	61.317	61.324	84.397	84.398	-23.08
NESMITH	41.514	41.526	63.923	63.923	-22.41
BICKFORD	52.329	52.342	75.072	75.072	-22.74
S714	43.628	43.619	66.477	66.464	-22.85
N99RESET	53.330	53.352	75.406	75.430	-22.08
J99	53.891	53.921	76.421	76.442	-22.53
Y683	71.995	72.000	94.600	94.600	-22.60
BEEF	120.518	120.526	142.914	142.914	-22.40
PRICE	113.610	113.621	135.645	135.646	-22.04
G287	179.954	179.957	202.088	202.076	-22.13
J54	63.700	63.695	86.267	86.253	-22.57
T714	46.759	46.778	69.481	69.481	-22.72
CORVA	103.478	103.473	126.017	126.017	-22.54
D728	56.099	N/A	79.168	N/A	-23.07
PEAV	106.054	106.042	128.639	128.618	-22.59
MAG	47.421	47.434	70.096	70.096	-22.68
Z714	148.861	148.851	171.152	171.126	-22.29
Q388RESET	51.906	51.916	74.659	74.661	-22.75
PEAK	585.359	585.323	607.584	607.539	-22.23

* Ellipsoidal height: NAD1983(2011) Epoch 2010.00
^ Elevation: NAVD1988

Table 23: STAR*NET Error Ellipses

Station	Ellipsoid ht 95% Confidence (cm)	Elevation 95% Confidence (cm)	Semi-Major Error Axis (cm)	Semi-Minor Error Axis (cm)	Azimuth of Major Axis (deg-min)
U727	1.60	Constrained	0.57	0.46	6-29
G728	1.27	1.19	0.49	0.42	3-13
NESMITH	1.82	C	0.70	0.55	3-34
BICKFORD	Constrained	Constrained	Constrained	Constrained	Constrained
S714	1.22	0.79	0.47	0.38	3-26
N99RESET	1.34	0.87	0.50	0.41	4-15
J99	1.89	1.43	0.67	0.53	5-41
Y683	1.47	Constrained	0.54	0.44	4-05
BEEF	1.40	0.94	0.55	0.44	4-06
PRICE	1.87	1.61	0.71	0.56	4-48
G287	1.67	1.37	0.61	0.49	3-54
J54	1.79	1.67	0.68	0.54	0-41
T714	1.67	Constrained	0.60	0.49	6-04
CORVA	1.80	Constrained	0.64	0.51	2-51
D728	1.81	1.76	0.60	0.51	178-57
PEAV	1.62	1.23	0.60	0.49	5-31
MAG	1.29	Constrained	0.48	0.38	3-43
Z714	1.50	1.22	0.51	0.41	2-59
Q388RESET	1.18	1.13	0.49	0.43	179-56
PEAK	1.91	1.59	0.71	0.58	4-08

Overall, the project fits within the 2 cm vertical standard in regards to the 95% confidence interval for all points. Also, most all of the solutions are within 2 cm of each other for both the STAR*NET and OP solutions. These results show that the 2 cm is easily achieved when following the guidelines. The worst vertical 95% confidence value for the study was 1.91 cm at the newly set station PEAK. The data was also much more precise in terms of horizontal coordinates. The average horizontal semi-major error ellipse axis was only 5.9 mm with a range from 4.7 mm to 7.1 mm.

7 DISCUSSION

7.1 COMPARING TRADITIONAL NETWORK PROCESSING AND OP

Overall, the traditional network processing conducted using LGO and STAR*NET produced very similar results to the central hub design of OP. Both techniques contain similarities and differences and each has their own strengths and weaknesses. Ultimately, a surveyor could use either method based on their preferences to obtain accurate network results. OP has some additional advantages when used in the central hub design that will be discussed in the recommendations for future research.

On average, the results compared to within 4 mm between STAR*NET and OP for ellipsoidal heights. For elevations, the resulting coordinates matched on average within 5mm and if the mutually constrained marks were excluded from the calculations to within 7mm. These results demonstrate that following NGS 58 and 59 produces results within 2 cm at 95% confidence in most cases.

Some differences did occur between the two post-processing techniques. First, for ellipsoidal results, three of the marks were greater than 2 cm from the STAR*NET results. Two of these marks included PEAK and J99 which contained signs of imprecise data on their OPUS-Static solutions. These stations had additional baselines and additional sessions in the traditional STAR*NET network that were not included in the OP processing due to the 2 hour session requirement. This allowed some of the error to be smoothed out for the STAR*NET results. The central hub design and the way that the multiple IGS stations fix the hub essentially limits station errors propagating to each respective station. Therefore, OP seems highly useful in determining which stations had errors, but STAR*NET's results for stations with observation errors may be closer to the ground truth due to the reduction of errors via its least squares adjustment.

The traditional network design has several strengths that include: full user control on network design, the ability to utilize shorter sessions within the network, and greater flexibility on including or excluding problematic baselines. The weaknesses of the traditional design include: software costs, additional work hours to process, and cumbersome nature of adding CORS or other data. Additionally, NGS 58 and 59 was written to exclude problematic baselines and to include shorter observations.

OP also has unique strengths and weaknesses. The open source and low cost of the program clearly makes it advantageous. The system also is relatively quick to process data and user friendly. OP's design also allows for the easy detection of imprecise data that may be inaccurate, and OP includes real world data that could reduce problems of static control in the traditional network setup. The central hub design with tight control also produces results that are not heavily influenced by errors at other marks and results in more accurate results for most stations. Weaknesses of OP include: not being able to process sessions shorter than 2 hours, limited user network design options, inability to process GLONASS data, and the lack of error ellipses and more robust statistical presentation of the results.

7.2 Trends and Mark Analysis

Some published marks seem to be outside of their published values. For elevations, G728 is also out of tolerance for both adjustments. STAR*NET also shows D728 greater than 2 cm, but this station was not available in OP due to less than 2 hour observations at that station. This 728 leveling line may have been mis-leveled and the only true way to verify its discrepancy would be to conduct precise geodetic leveling. BEEF had the largest ellipsoid difference from published both in STAR*NET and OP. U727 also had ellipsoid heights greater than 2.5 cm from published for both OP and STAR*NET. All of these marks seem to portray data consistently off from their published coordinates. Regrettably, both G728 and U727 were primary marks in the network and were Cooperative Base Network stations included in the development of GEOID12A. It is unknown if their published elevations or ellipsoid heights were originally bad or if the marks moved at some point in time.

Two marks are also clearly incorrect in regards to northing and easting. Both adjustments show J54 as different in northing and easting and OP also shows its elevation greater than 2 cm. These outlying values along with the appearance of J54 suggests that it was indeed disturbed. Station PRICE is also off from published by a value of 1.8 cm in northing and 1.7 cm in easting for OP and greater than 2 cm away from published for both values in STAR*NET. This station is located near a road drainage ditch and subsidence or mark movement may have occurred over time.

According to NGS geoid accuracy maps, the local geoid model that was derived from these processing techniques fits within the error budget of the GEOID12A model of 3-4 cm at 95%. In fact, the new locally derived geoid model was on average 6.6 mm in difference from GEOID12A with a high difference of only 1.28 cm. This trend suggests that the GEOID12A is relatively accurate within the project's site. In fact, the models agreed so closely that the points used to model GEOID12A showed no unique trend in geoid accuracy compared to marks that were not used for GEOID12A's modeling. The predicted elevations when using GEOID12A as a correction to the derived ellipsoid heights was also relatively close to the final orthometric adjustment, but this practice is not recommended. The accuracy of GEOID12A is not always as precise at other project sites. Because of this, a complete network is needed for more accurate results until the error within the geoid model is greatly reduced.

Trends within height errors between B stability and C stability marks were also analyzed. The average differences from the published values for both ellipsoid heights along with standard deviations were calculated. Also, the number of benchmarks that exceed 2 cm from published or the marks considered valid during the orthometric adjustment were also tabulated. Results from the STAR*NET solution (Table 24) and the OP solution (Table 25) were compared for trends.

Table 24: B vs. C Stability STAR*NET Results

Station	**Change h_{83} from published (cm)**	**Change H_{88} from published (cm)**	**Stability**
U727	2.59	Constrained	B
G728	0.77	-2.39	B
S714	2.53	1.25	B
T714	0.71	Constrained	B
CORVA	0.70	Constrained	B
D728	N/A	-2.47	B
Z714	N/A	2.61	B
NESMITH	-0.46	Constrained	C
BICKFORD	Constrained	Constrained	C
N99RESET	-1.44	-2.36	C
J99	-0.66	-2.1	C
Y683	0.52	Constrained	C
BEEF	2.70	N/A	C
PRICE	0.78	N/A	C
G287	1.48	1.18	C
J54	-0.11	-1.77	C
MAG	N/A	Constrained	C
Q388RESET	N/A	N/A	C
Average B	1.46	-0.25	
Std Dev B	1.005	2.578	
# > 2 cm B	2	3	
# valid ortho B	N/A	3	
Average C	0.35	-1.26	
Std Dev C	1.315	1.646	
#> 2 cm C	1	2	
# valid ortho C	N/A	4	

Table 25: B vs. C Stability OP Results

Station	Change h_{83} from published (cm)	Change H_{88} from published (cm)	Stability
U727	2.5	-1.0	B
G728	1.5	-2.3	B
S714	1.6	Constrained	B
T714	2.6	Constrained	B
CORVA	0.2	Constrained	B
Z714	N/A	Constrained	B
NESMITH	0.7	Constrained	C
BICKFORD	1.3	Constrained	C
N99RESET	0.8	Constrained	C
J99	2.3	Constrained	C
Y683	1.0	Constrained	C
BEEF	3.5	N/A	C
PRICE	1.9	N/A	C
G287	1.8	Constrained	C
J54	-0.6	-3.2	C
MAG	N/A	Constrained	C
Q388RESET	N/A	N/A	C
Average B	1.52	-1.65	
Std Dev B	0.954	0.919	
# > 2 cm B	1	1	
# valid ortho B	N/A	4	
Average C	1.50	-3.20	
Std Dev C	1.200	N/A	
#> 2 cm C	2	1	
# valid ortho C	N/A	7	

These results do not show a strong discrepancy between B stability and C stability marks, and both seem to be relatively similar. The 95% confidence values for ellipsoid heights calculated from STAR*NET were also considered for trends between B stability and C stability and are presented in Table 26 below.

Table 26: Stability vs 95% Confidence

Station	Stability	95% Confidence Ellipsoid Height (cm)
U727	B	1.60
G728	B	1.27
S714	B	1.22
T714	B	1.67
CORVA	B	1.80
D728	B	1.81
Z714	B	1.50
NESMITH	C	1.82
BICKFORD	C	Constrained
N99RESET	C	1.34
J99	C	1.89
Y683	C	1.47
BEEF	C	1.40
PRICE	C	1.87
G287	C	1.67
J54	C	1.79
MAG	C	1.62
Q388RESET	C	1.18
Average B		1.55
Std Dev B		0.237
Average C		1.61
Std Dev C		0.248

This data agrees to within just under 1 mm between the two stabilities from a 1.55 cm average 95% confidence window for stability B and a 1.61 cm average 95% confidence window for stability C. The residuals between B stations alone and C stations alone also did not portray any trends to suggest one type of monument is clearly more precise or accurate over another. Therefore, no conclusion can be made that stability class of the monuments influences the data.

Trends in precision were also checked compared to the level of overhead visibility at each of the stations. The stations were categorized by having high, medium, or low level of overhead visibility based on the visibility diagrams collected at each station. A table showing these overhead visibility categories and each stations' resulting ellipsoid height 95% confidence values are depicted in Table 27 below.

Table 27: Overhead Visibility vs 95% Confidence

Station	Overhead Visibility	Ellipsoid Height 95% Confidence (cm)
G728	HIGH	1.27
NESMITH	HIGH	1.82
BICKFORD	HIGH	Constrained
N99RESET	HIGH	1.34
Y683	HIGH	1.47
D728	HIGH	1.81
U727	LOW	1.60
J99	LOW	1.89
PRICE	LOW	1.87
CORVA	LOW	1.80
MAG	LOW	1.29
PEAV	LOW	1.62
S714	MEDIUM	1.22
BEEF	MEDIUM	1.40
G287	MEDIUM	1.67
J54	MEDIUM	1.79
T714	MEDIUM	1.67
Z714	MEDIUM	1.50
Q388RESET	MEDIUM	1.18
PEAK	MEDIUM	1.91
Average High		1.54
Std Dev High		0.260
Average Medium		1.54
Std Dev Medium		0.264
Average Low		1.68
Std Dev Low		0.227

This data does show a slight increase in the 95% confidence widows as one observes at stations with a low overhead visibility. Overall, the error ellipses are close in magnitude for these categories and additional research with more stations and data is needed to ascertain the exact effects of varying overhead obstruction.

Stations with overhead powerlines were also examined for any trends in influence on precision. All of the stations containing powerlines are presented in Table 28. The average 95% confidence region for ellipsoid height was 1.6 cm from stations with powerlines present and was also 1.6 cm for stations without powerlines present with a change from the two of only 0.4 mm. Both of these groups also had fairly similar standard deviations with the stations without powerlines having only a slightly smaller standard deviation. Therefore, the presence of powerlines seems to have little effect on the data. If the powerlines do in fact effect the data, then the long observations conducted in this study mitigated their effect to a minimum.

Table 28: Effect of Overhead Powerlines on Precision

Station	Powerlines	95% Confidence Ellipsoid Height (cm)
G728	Yes	1.27
NESMITH	Yes	1.82
J99	Yes	1.89
CORVA	Yes	1.80
S714	Yes	1.22
G287	Yes	1.67
J54	Yes	1.79
Z714	Yes	1.50
BICKFORD	No	Constrained
N99RESET	No	1.34
Y683	No	1.47
D728	No	1.81
U727	No	1.60
PRICE	No	1.87
MAG	No	1.29
PEAV	No	1.62
BEEF	No	1.40
T714	No	1.67
Q388RESET	No	1.18
PEAK	No	1.91
Average Yes	1.62	
Std Dev Yes	0.260	
Average No	1.56	
Std Dev No	0.246	

The difference from published for orthometric benchmarks was also compared to different leveling vertical orders. The changes from published for each mark divided into first, second, and third order leveling for the minimally constrained adjustment ran in STAR*NET is presented in Table 28 and shows a slight correlation in closeness to published elevation compared to more precise leveling. First order leveling differed on average by 1.2 mm away from published while Second order leveling differed by 5.1 mm. Lastly, third order differed by 8.5 mm.

Table 29: Vertical Order vs. Differences from Published Elevation

Station	**Change H_{88} from published (cm)**	**Vertical Order**
U727	0	First
G728	-2.01	First
S714	1.49	First
T714	0.09	First
CORVA	Constrained	First
D728	-2.11	First
Z714	2.69	First
BICKFORD	0.51	First
J54	-1.61	First
MAG	-0.17	First
J99	-2.25	Second
G287	1.24	Second
NESMITH	-0.72	Third
N99RESET	-2.36	Third
Y683	0.54	Third
Average First	-0.12	
Std Dev First	1.609	
Average Second	-0.51	
Std Dev Second	2.468	
Average Third	-0.85	
Std Dev Third	1.454	

This same trend is not present in OP's results. The results of the minimally constrained orthometric adjustment conducted in OP associated with each mark's vertical order is presented in Table 29. These results do not show a trend between the different leveling orders with First order having an average difference from published of 6.1 mm, second order a difference of 0.5 mm, and third order a difference of 6.0 mm.

Table 30: OP Elevation Differences from Published vs. Vertical Order

Station	**Change H_{88} from published (cm)**	**Vertical Order**
U727	-1.1	First
G728	-2.4	First
S714	-0.4	First
T714	0.9	First
CORVA	Constrained	First
Z714	0.7	First
BICKFORD	0.6	First
J54	-3.2	First
MAG	0	First
J99	-0.5	Second
G287	0.6	Second
NESMITH	-0.6	Third
N99RESET	-1.1	Third
Y683	-0.1	Third
Average First	-0.61	
Std Dev First	1.513	
Average Second	0.05	
Std Dev Second	0.778	
Average Third	-0.60	
Std Dev Third	0.500	

7.3 RESEARCH KEY FINDINGS

By following NGS 58 and 59 it is clear that 2 cm at 95% confidence accuracy can be achieved in most circumstances. For this study, elevations were determined to ± 0.8 – 1.8 cm at 95% confidence. When comparing the results of LGO and STAR*NET's processing of the network compared to OP, the results also agree within 2 cm on most cases and usually always agrees within 3 cm except in cases of known multipath. Therefore, the NGS claim that following NGS 58 and 59 allows for the attended accuracies "almost always" was confirmed.

Today's improvements in GPS receivers and antennas along with increased number of satellites led to accurate results, but today's newer post-processing software may be the biggest difference between processing networks today compared to the late 1990's. For example, OP is capable of similar 2 cm accuracy, but the results in OP depend on particular user practices. In fact, this study demonstrates that OP can achieve similar elevation coordinates as traditional methods and these results compared within 5 mm.

A key finding in this research was certain practices in OP increased accuracy and precision when compared to both published coordinates and solutions derived in STAR*NET. The results of this study led to the following recommendations for post-processing in OP. First, the central hub design utilizing a 24 hour station should be used for the network design. Additionally, when the hub is fixed by control stations that are held tight, the baseline data of the local stations have little effect on propagating error to each other. Lastly, not all CORS stations are equal in precision and accuracy, and the results of this study suggest incorporating IGS stations or even using only IGS stations as control. The increases in accuracy resulting from incorporating IGS stations may be due to their increased regulation, more accurate coordinates, and the combined benefits of resolving tropospheric delay with longer baselines.

The effects of OP and its utility presents several areas of future research that may greatly improve efficiency in conducting height modernization surveys. Some of these recommendations for future research are presented in the next section. Additionally, some recommendations on improving the guidelines of NGS 58 and 59 are presented in section 7.5.

7.4 RECOMMENDATIONS FOR FUTURE RESEARCH

Specifics of OP require additional research. For instance, accuracy similarities using only 2 hours of data within OP but still following a multi-day and time of day campaign should be compared with a network containing longer sessions. The shorter two hour sessions with the same 24 hour central hub technique with tight control may result in very similar accuracies to longer sessions and reduce the need for longer static observations. This area of research was not fully developed due to time. Given the stability of the central hub design, research may show that the 2 hour observations may be within the intended 2 cm accuracy range for ellipsoidal and orthometric heights.

Initial testing also showed that changing the mask angle had effects on the ultimate solution. In most cases, increasing the mask angle within OP resulted in values closer to published, but this fact was not consistently true for all stations or sessions. Network solutions even dropped to a level of 6 mm difference from the published heights when changing the mask angle unique for each session. The reason for a possible increase in accuracy is due to a decrease in multipath and GPS signals containing more atmospheric errors influencing the solution. However, the drawback is that less overall data is used and sometimes useful data is neglected when the mask angle is increased. Research may show that 15 degrees is not the ideal mask angle and sites with overhead obstructions may require different mask angles for more accurate results.

Slightly related to mask angle is the amount of overhead obstruction and its effects on the solution. This area of research can help to make recommendations on how much overhead obstruction is significant and how much overhead obstruction influences the final results. A combination of obstructed marks along with open marks or partially obstructed should be tested in a scientific study. The results may suggest a percent of sky obstructed or degrees from the horizontal of an obstruction that would exceed recommended for height modernization surveys.

Including GLONASS data is another area for future research. Although OP lacks a GLONASS processing feature, studies should be conducted on traditional network designs with the addition of GLONASS. These additional satellite observations may increase accuracy, and this type of study should be combined with the earlier mentioned mask angle and overhead obstruction studies to see if GLONASS improves results in obstructed scenarios.

Perhaps the biggest utility of OP is the central hub network design and its relative ease of use. One area of future research that may further demonstrate the power of OP is a comparison of northing, easting, and ellipsoidal results with a smaller network or no local network at all. Since the ultimate determination of a static mark's coordinates depends solely on the position of the central hub fixed by tight control and the relative baselines between the two, then in theory, a network of statically observed marks may not be needed to determine the ellipsoid heights and horizontal coordinates of a single station. This is due to the fact that errors and values of other stations do not influence each other to a large extent. Therefore, research should be conducted as to how many 2 hour observations of a single station can produce similar results to an entire network like the one used for this study. This single station should still be differenced in OP within 40 km to a 24 hour central hub. This hub should be positioned by 24 hour IGS control stations held tight. If this theory proves true, then fewer overall stations and dual observations between stations may be required in order to determine ellipsoidal heights and horizontal coordinates. This same theory does not hold true for orthometric heights due to the requirement to include published elevations within 20 km for a complete NGS 59 processing.

Research concerning the impacts of trivial vectors on final accuracy should also be conducted. Session planning on identifying trivial and independent vectors is time consuming. If changes in accuracy when including trivial vectors is minimal, then perhaps networks can be designed to keep all possible vectors. Impacts of imprecise or erroneous data on a network that includes all possible vectors may also change the network design requirements of NGS 58 and 59.

An investigation on why OP produces unreasonable error estimates should also be conducted. In this study the average error in ellipsoid height was usually 1-2 mm with one station at 6 mm. These values do not seem to represent real world error ranges for GPS surveys. Further research on the specifics on how OP derives its error results may lead to refinement and ultimate production of more accurate error statistics.

7.5 Recommendations For Improving NGS 58/59

Additional research is needed to make complete recommendations for improving or replacing NGS 58 and 59 guidelines. However, some aspects might be altered. First, key requirements that should be included are repeat observations at every mark, and observing different observations at different times of the day in order to observe different satellite geometries. These aspects are needed in order to check for errors and ensure that the different geometry and atmospheric solutions are utilized for more accurate results.

The recommendation for 20 km spacing of valid orthometric marks was not fully tested, and this recommendation is still most likely needed for orthometric processing. In fact, more orthometric marks included in a project will increase the final accuracy of elevations of unknown stations. Due to the results of the central hub and the recommendations for future research mentioned above, it may be possible to derive accurate ellipsoidal heights without as robust of a network. Therefore, the guidelines for establishing ellipsoidal heights in NGS 58 may need to be changed to suit a more efficient central hub survey. The guidelines for NGS 59 should also be re-written to include the execution steps outlined in NGS 58 which are still needed for orthometric processing.

Changes could also occur to the time requirements for observations such as the initial 5 hour sessions for three days. With the use of the central hub technique utilizing a 24 hour hub linked to IGS stations, it seems that the 5 hour requirement may be unnecessary. In fact, the requirement for NSRS and primary stations may be entirely outdated as long as the central hub is within 40 km of all secondary and local marks. If a 24 hour station is in fact within 40 km, then the network could be processed to the same degree of accuracy as the traditional spacing of NSRS and primary stations, because the IGS or CORS will effectively locate the central hub which will serve as the only needed primary station. If the need for NSRS and primary observations are excluded, then the overall amount of survey time may be greatly reduced. The downside is that OP cannot process data less than 2 hours at each station. However, the guidelines of NGS 58 and 59 may be changed to eliminate the requirement for NSRS and primary stations.

NGS requirements to conduct detailed meteorological readings before, during, and after each station setup may be unnecessary. The purpose of recording this data is not explicitly for post-processing and intended to help identify moving weather fronts. The guidelines should be changed to make these readings a suggestion rather than a requirement.

NGS requirements for re-observations of repeat baselines with differences in Up > 2.0 cm and RMS > 1.5 cm appear to be arbitrary and should be replaced with a statistical test. These guidelines may work in some cases, but they are not a true test to check for outliers. In order to exclude outlying data properly, NGS should implement a statistical test such as excluding repeat baseline magnitudes that are 3 standard deviations or even 3.29 standard deviations away from the mean. A statistical test and process of removing outliers would ensure that actual real world noise is not inadvertently removed from the data.

8 CONCLUSION

Height modernization with GNSS is a practical alternative to traditional geodetic leveling for determining accurate elevations to the centimeter level. Even though geodetic leveling is currently more accurate, this study centered on the application of utilizing GNSS for determining heights due to increases in efficiency and reduction of costs. Specifically, this study evaluated NGS guidelines for height modernization put forth in NOS-NGS 58 and NOS-NGS 59 by focusing on modern advancements in receiver and antenna technology, today's more robust satellite constellations, and commercial post-processing software. Recommendations for modification and/or optimization of the guidelines were made, and the accuracy of OP was evaluated. Lastly, an objective was to provide recommendations for future research.

One objective to this study was to evaluate the accuracy of OPUS-Projects, and a key finding in this study is that OP can be used to find elevations similar to those found following NGS 58 and 59 guidelines along with commercial software. To do so, a user-defined network with a central hub tightly constrained to IGS or very stable CORS active stations is recommended. Also, 24 hour baselines are recommended for connecting the central hub to the active stations. In addition, very long baselines from the central hub to the active stations are recommended for resolving tropospheric modeling errors. Given the results of this study and modern improvements in GNSS-related technologies, the guidelines of NGS 58 and 59 could be optimized, as these guidelines are based on 1990s equipment and experiences. Future research is needed to further increase efficiency and cost savings of GNSS height modernization.

The accuracy of GNSS coupled with the efficiency of data collection clearly make modern height determinations via GNSS exciting, and evaluating the accuracy of NGS 58 and 59 was another major objective of this study. Improvements on accuracy and efficiency in determining heights via GNSS will result in additional cost savings for the scientific and engineering communities.

After conducting a ten-day static GPS survey in Oregon, and by closely following guidelines set forth in NGS 58 and 59, errors in elevations were determined to range from ± 0.8 to 1.8 cm at 95% confidence, and the average elevation error at 95% confidence was ±1.3 cm. Another benefit of performing GNSS height modernization is that very accurate horizontal coordinates are found at each survey station. Unlike geodetic leveling, GNSS yields 3D positions. During this study, errors in horizontal coordinates at each station were estimated to range from ±0.5 to 0.7 cm at 95% confidence.

This study demonstrated that the published coordinates at several of the passive marks have some error, and this fact highlights the need for routinely updating the coordinates on the passive marks by performing new GNSS height modernization surveys. The ability to quickly determine new control coordinates in 3D also alludes to the benefit of GNSS height modernization, especially given that NGS is no longer conducting costly geodetic control surveys, or maintaining passive marks.

Due to the ±0.8-1.8 cm range and ±1.3 cm average level of accuracy at 95% confidence achieved for elevations in this study and given the changes in GNSS from the 1990s, several recommendations for modification and optimization of NGS 58 and 59 were given. Determining these recommendations was also a major objective of this project. First, the requirements for the outer linking NSRS and Primary stations may be outdated given today's robust CORS and IGS networks. If a central hub is within 40 km of all marks in a survey, then stable IGS or CORS active stations can be used as constraints for determining the latitude, longitude, and ellipsoid height position of the central hub. Therefore, establishing a survey hierarchy (i.e., designating primary and secondary marks) may be un-needed if a suitable central hub is in the project area. Although a user-defined network with a central hub reduces many of the spacing and network design requirements within NGS 58, NGS 59's requirement for the spacing of valid NAVD88 benchmarks is still needed to accurately determine orthometric heights. Other principles that should remain in any GNSS height modernization survey include observing on different days and different times of the day. These principles help to more fully model the random error in the GNSS measurements.

OP's ease of use and low cost make it very attractive to the surveying community, and determining recommended practices and relative accuracy of

OP was a key aspect of this study. Clearly, post-processing networks in OP allows users to quickly and easily derive heights compared to following traditional NGS 58 and 59 post-processing techniques.

The last objective of this study was identifying several areas requiring future research. First, the testing of only 2 hours of data within OP and its resulting accuracy compared to network solutions using more hours of data should be explored. Unfortunately, this could not be tested due to a lack of data. Second, additional research on the ideal mask angle and the effects of changing the mask angle should be conducted. Third, the incorporation of GLONASS and its effects on accuracy, especially in obstructed areas, should be tested. Lastly, comparing the accuracy of a single point differenced to a central hub network verses a complete network of local points differenced to a central hub may reduce overall network requirements put forth in NGS 58 and 59. Additional research in this area may lead to drastic savings in regards to survey execution time and costs.

Ultimately, NGS 58 and 59 could be altered to represent modern day improvements in technology such as refined GPS receivers and improved post-processing software along with incorporating the robust CORS and IGS networks. Further research in height modernization and in OP may yield additional savings as the surveying community prepares itself for the eventually high accuracy geoid model and new gravity-based vertical datum that is currently due to be released by NGS after 2022. As technology improves, models are refined, and post-processing software develops, the ability to determine orthometric heights via GNSS may someday reach the sub-centimeter level.

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APPENDICES

APPENDIX A – STATION PHOTOS

Station BEEF

Facing West

Station BICKFORD

Facing South

Station CORVA

Facing West

Station G287

Facing West

Station J54

Facing North

Station MAG

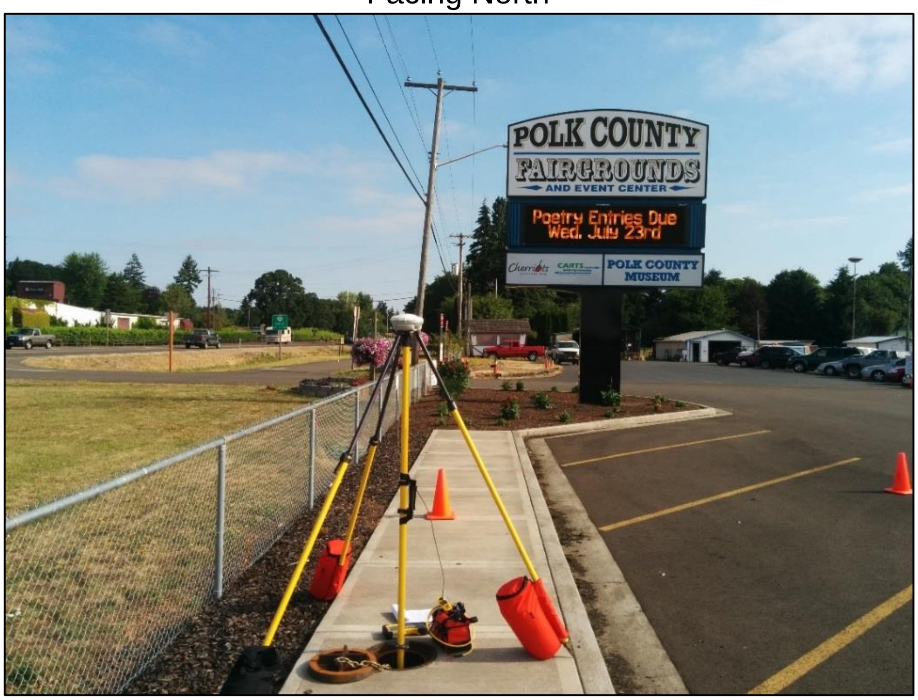
Facing West

Station N99-RESET

Facing South

Station NESMITH

Facing North



Station PEAK

Facing East

Station PEAV

Facing West

Station PRICE

Facing West

Station Q388 RESET

Facing South

Station S714

Facing North

Station T714

Facing North

Station U727

Facing North

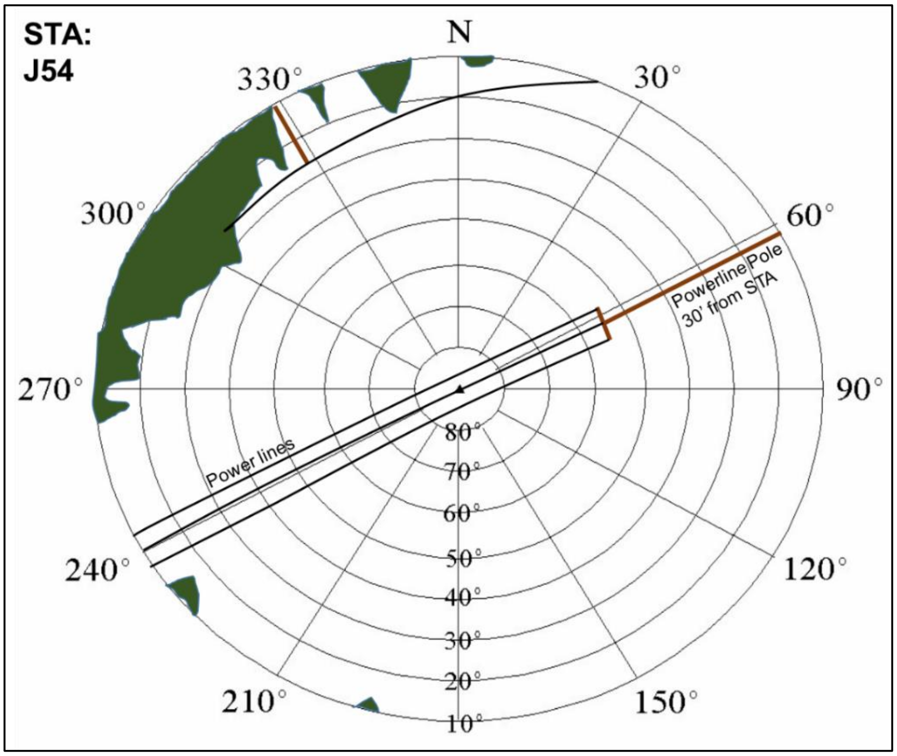
Station Y683

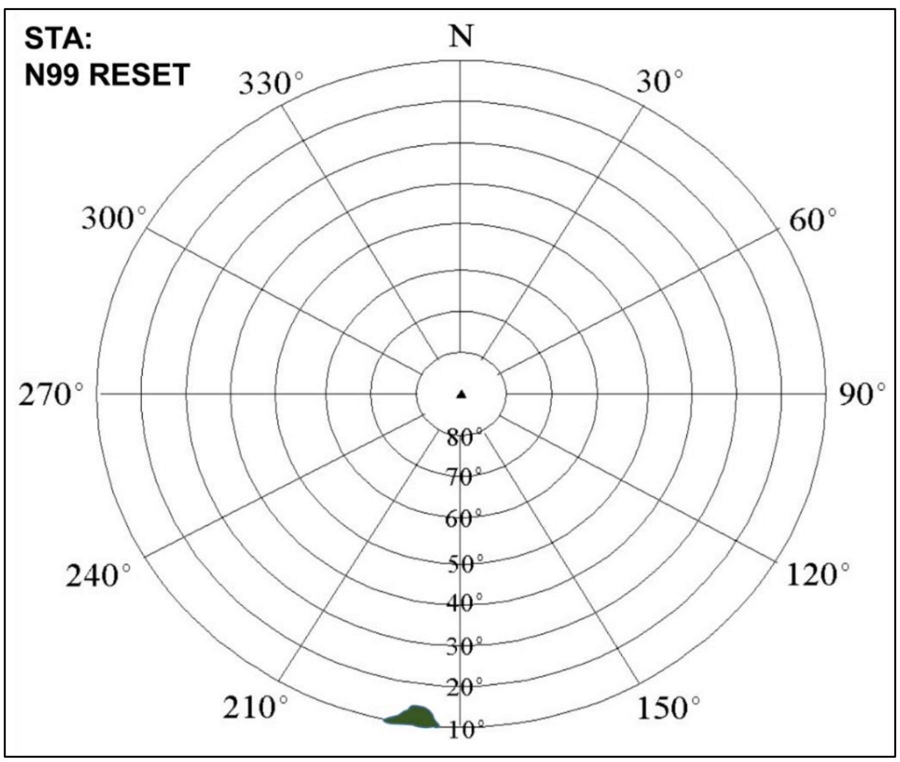
Facing North West

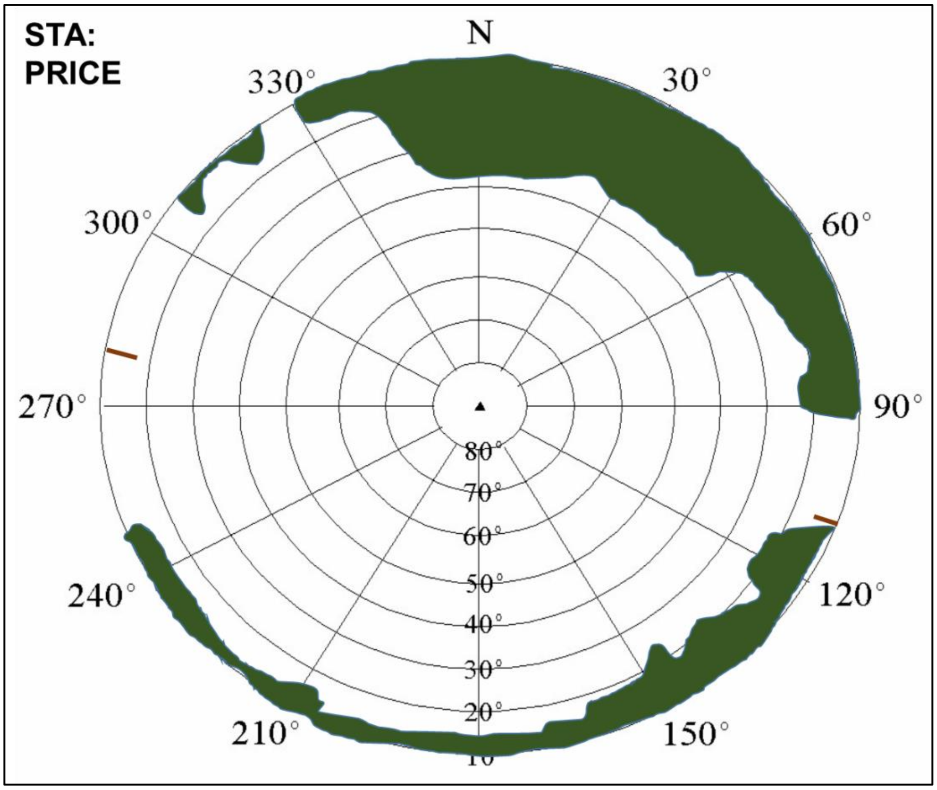
Station Z714

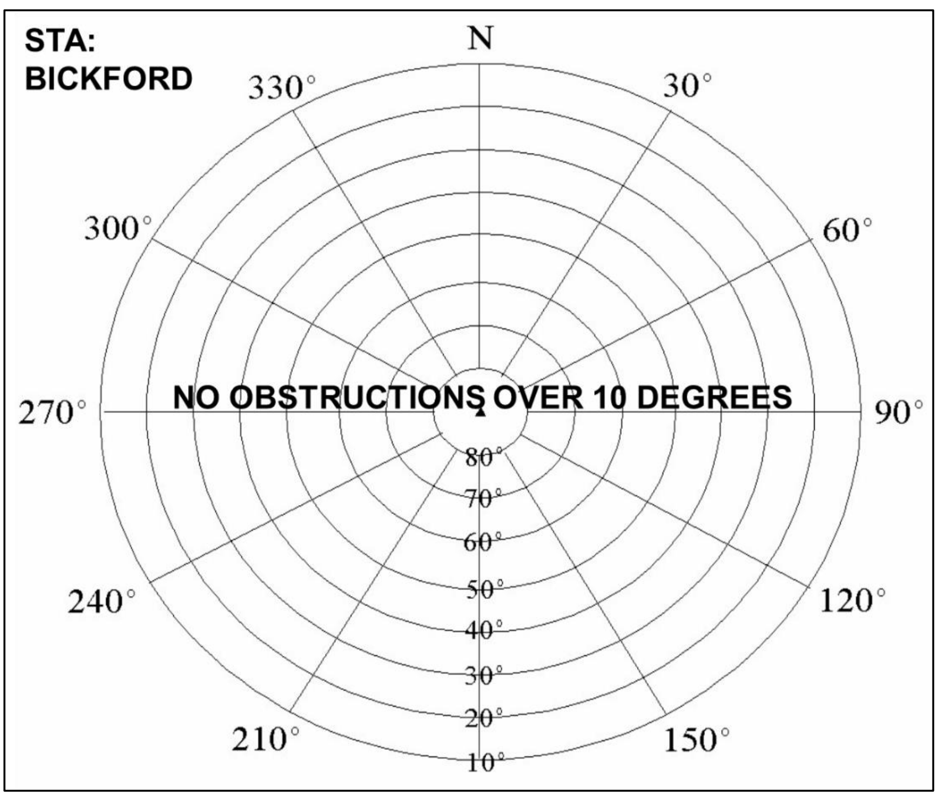
Facing North

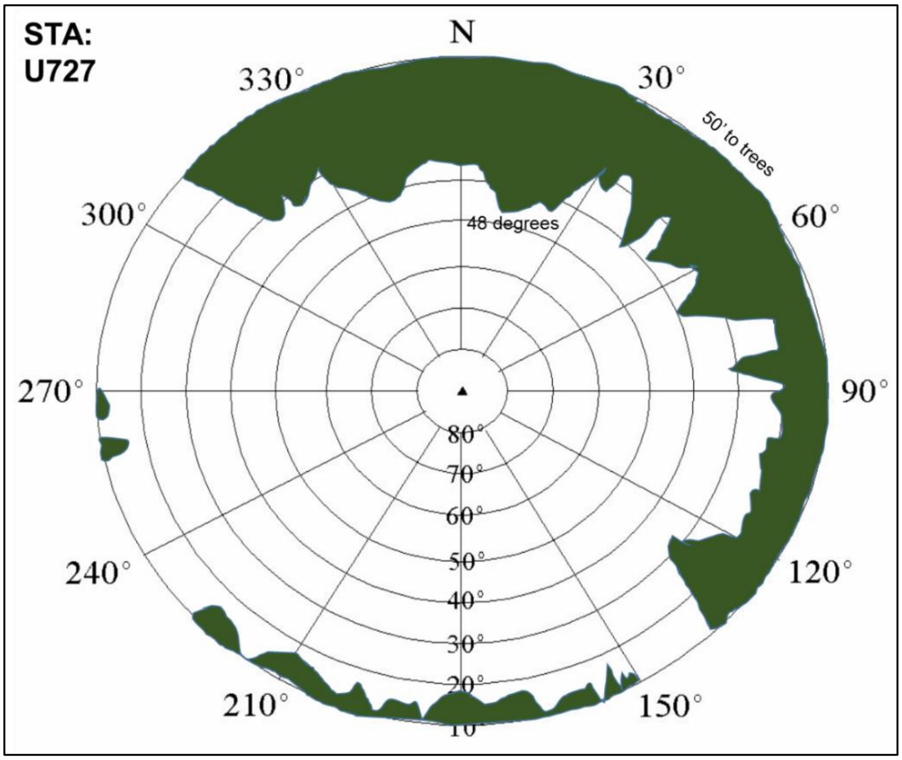
APPENDIX B – VISIBILITY DIAGRAMS

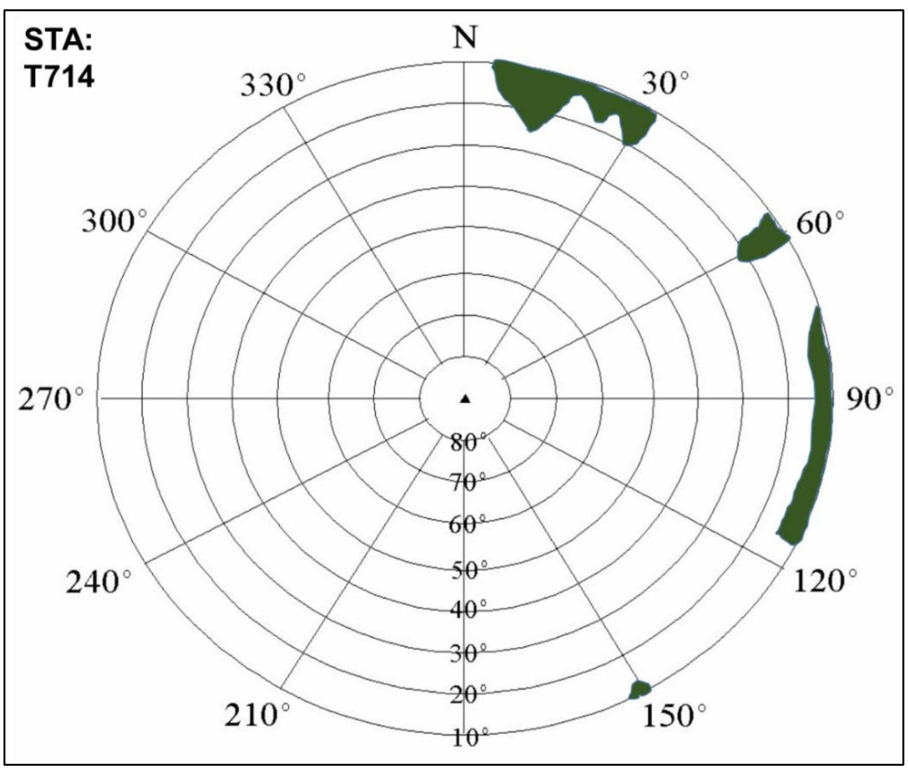

STA:
J54
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°
Power lines
Powerline Pole
30' from STA

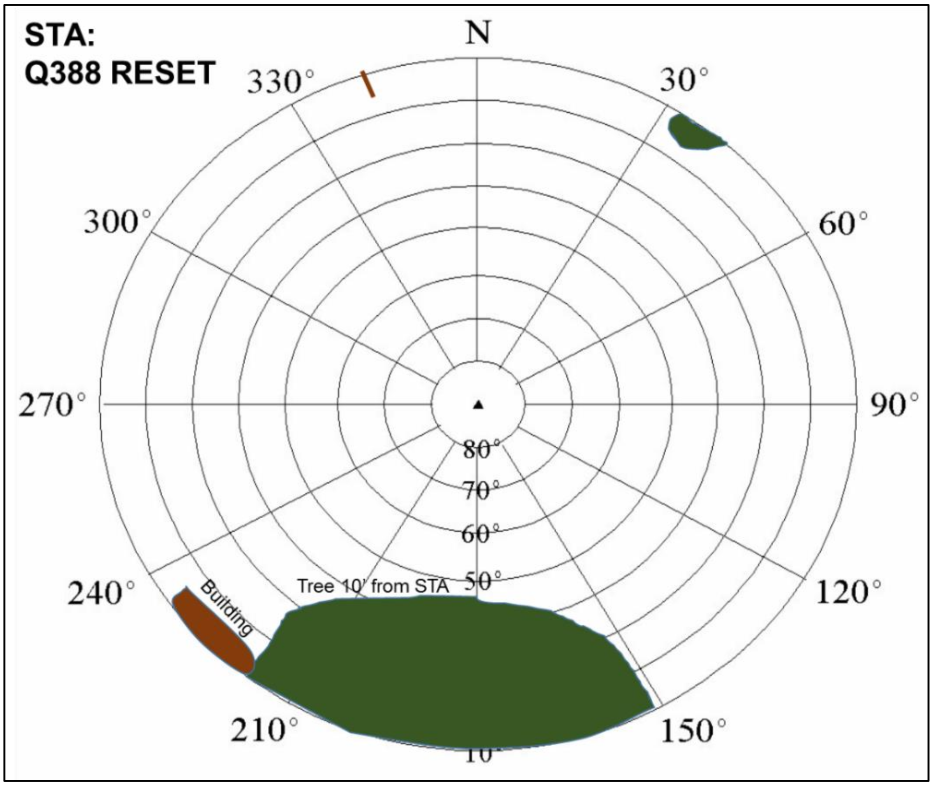

STA:
N99 RESET
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°

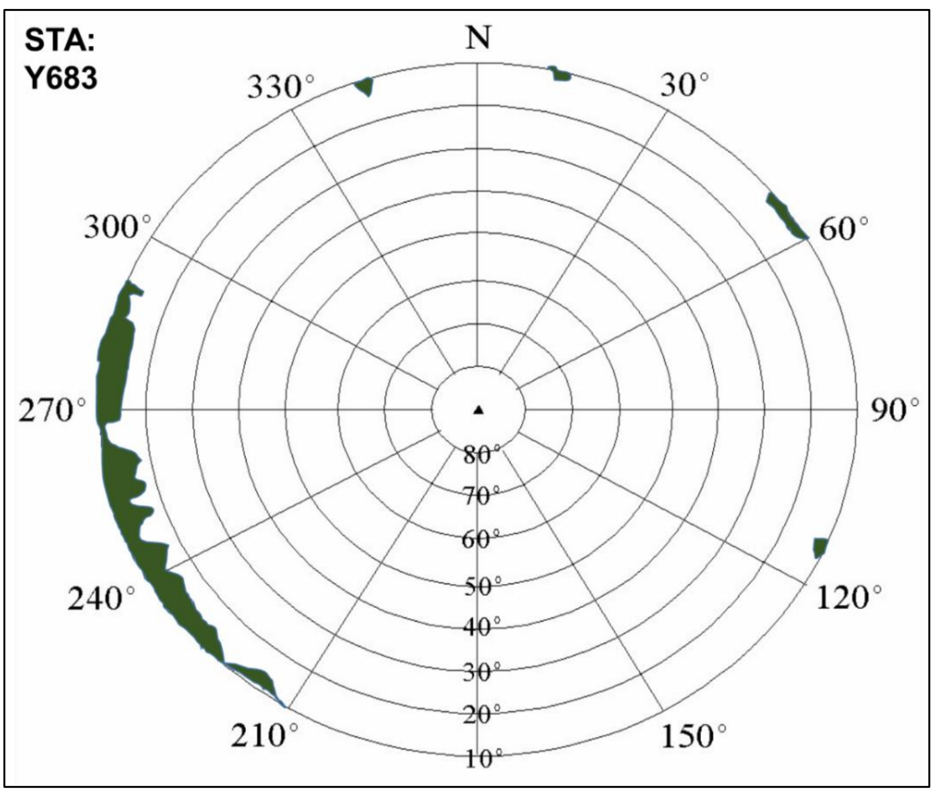

STA:
PRICE
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°

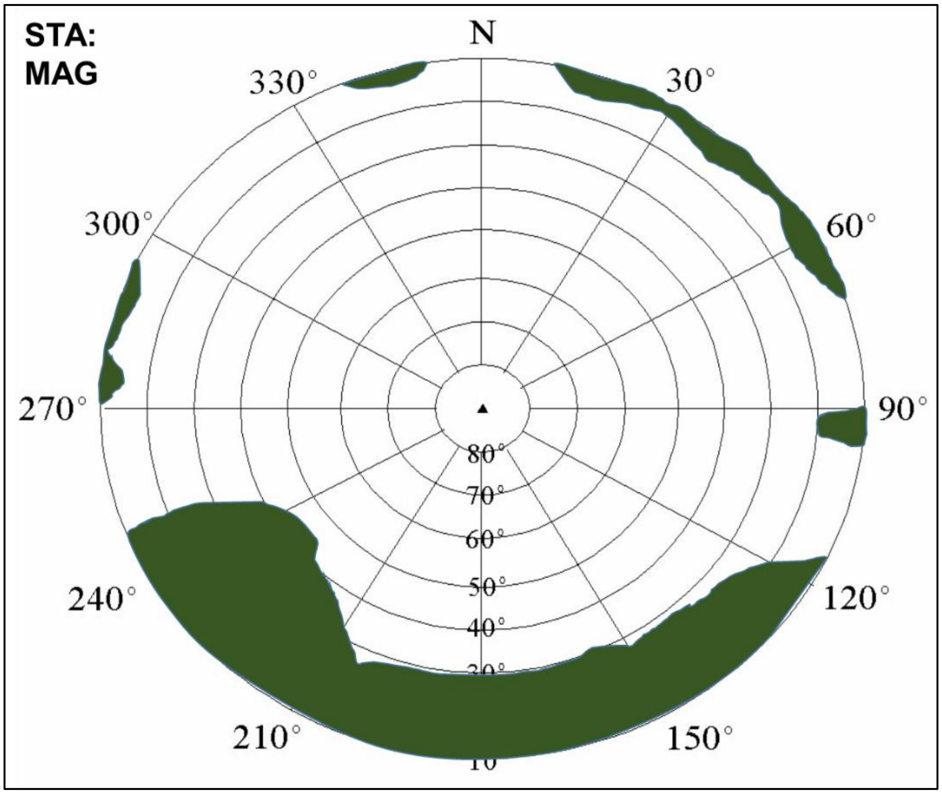

STA:
BICKFORD
N
330°
30°
300°
60°
270°
NO OBSTRUCTIONS OVER 10 DEGREES
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°

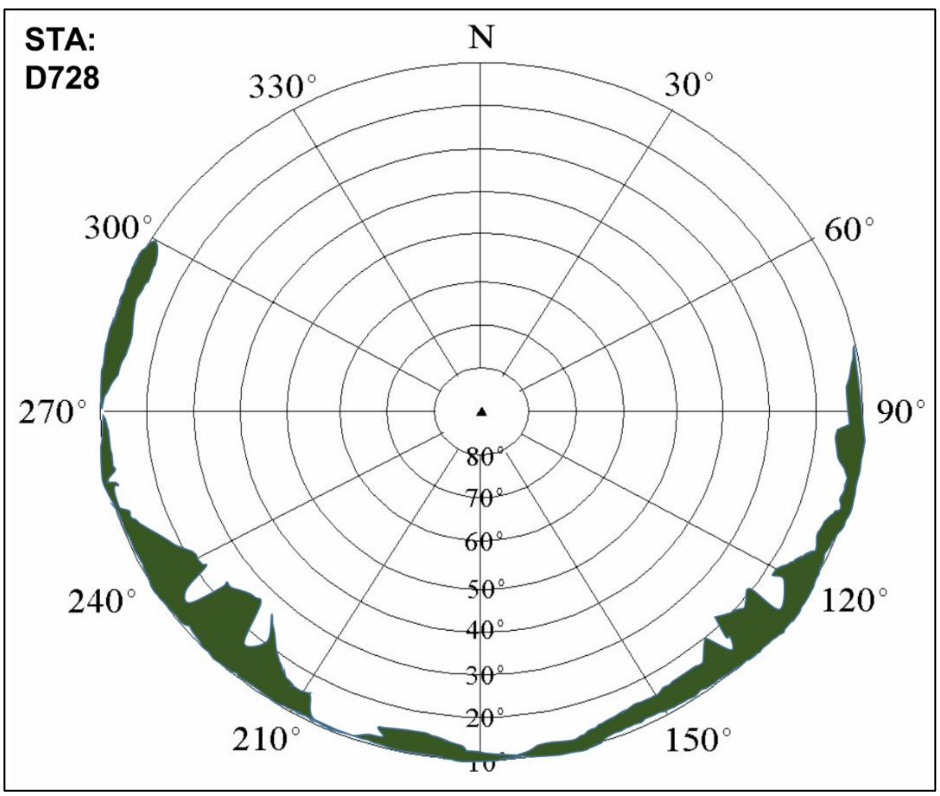

STA:
U727
N
330°
30°
50' to trees
300°
60°
48 degrees
270°
90°
80°
70°
60°
50°
240°
120°
40°
30°
20°
210°
150°
10°

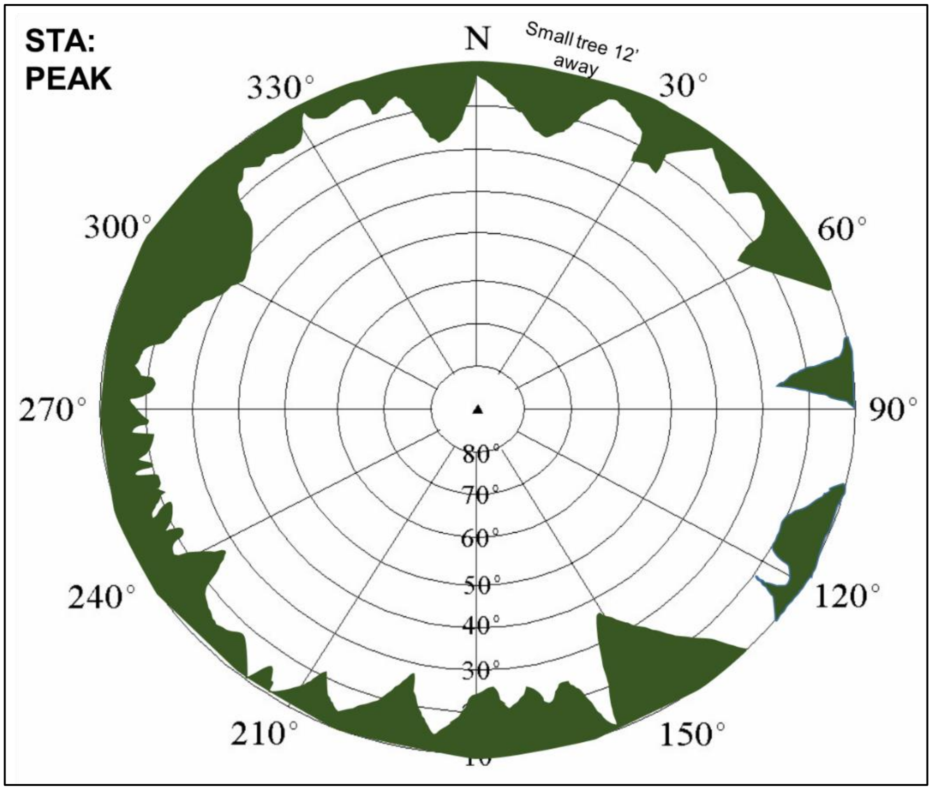

STA:
T714
N
330°
30°
300°
60°
270°
90°
80°
70°
60°
50°
240°
120°
40°
30°
20°
210°
150°
10°

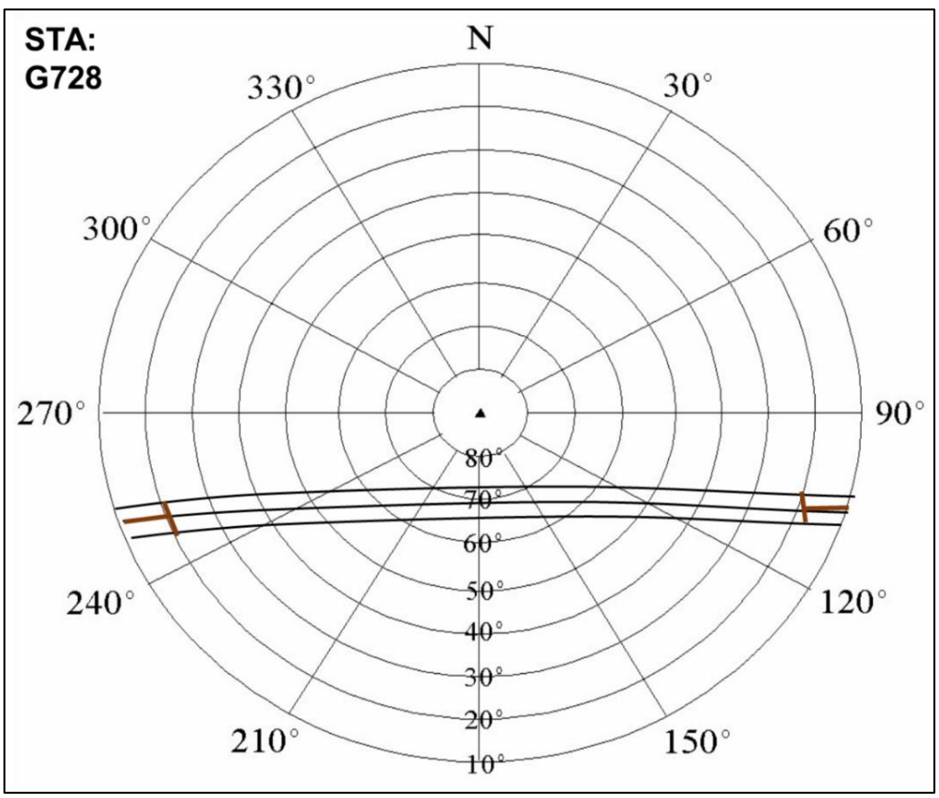

STA:
Q388 RESET
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
Building
Tree 10' from STA

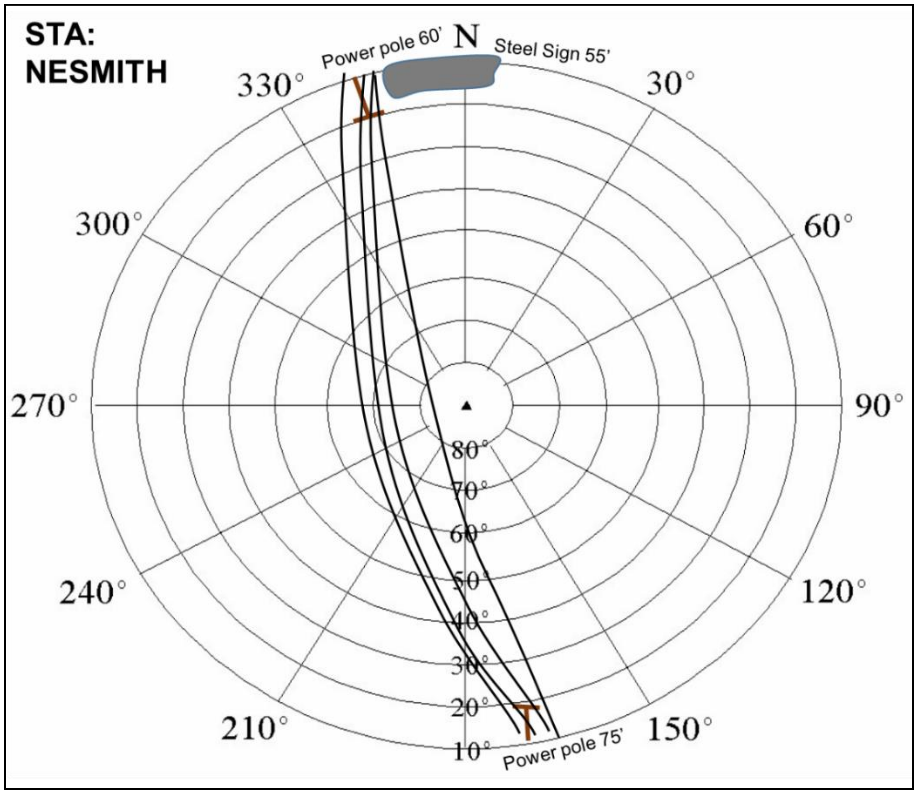

STA:
Y683
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°

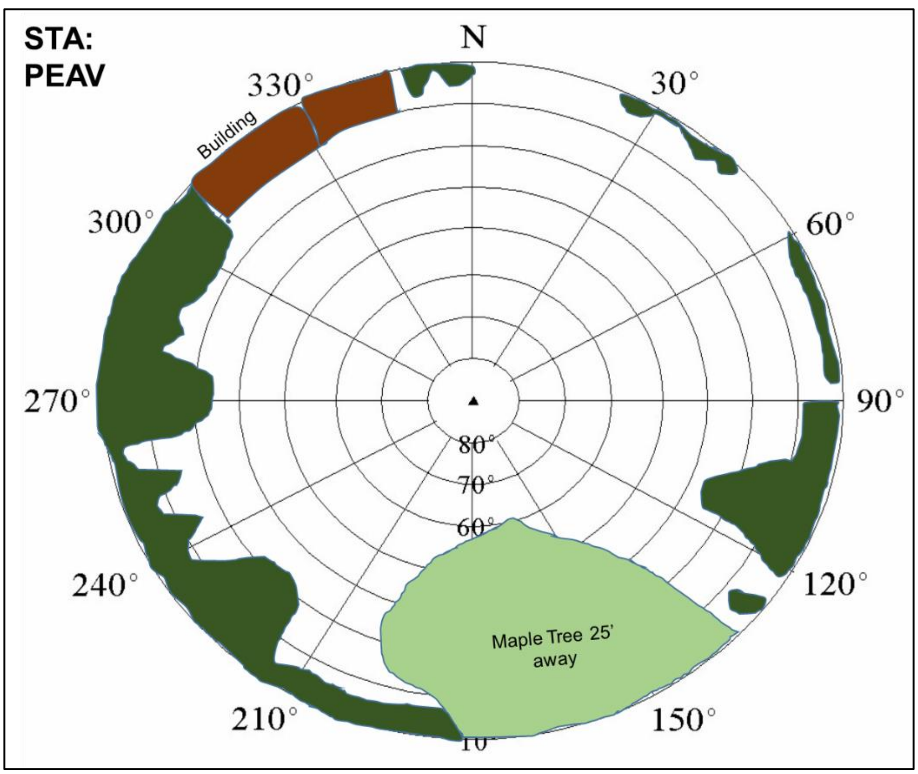

STA:
MAG
N
330°
30°
300°
60°
270°
90°
80°
70°
60°
50°
40°
240°
120°
210°
150°

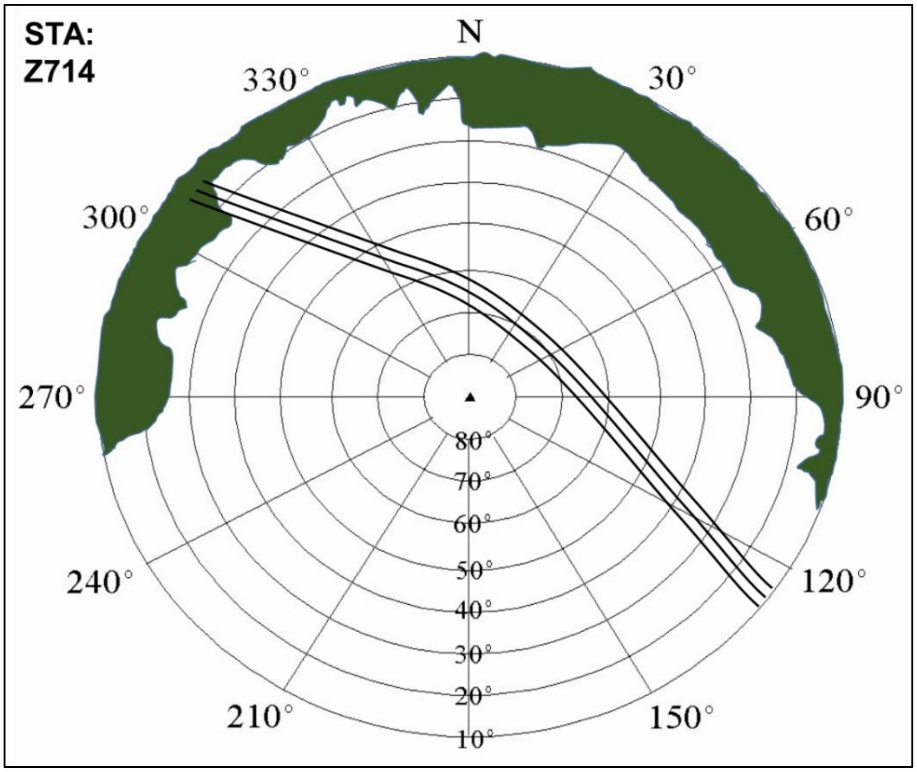

STA:
D728
N
330°
30°
300°
60°
270°
90°
80°
70°
60°
50°
40°
30°
20°
240°
120°
210°
150°

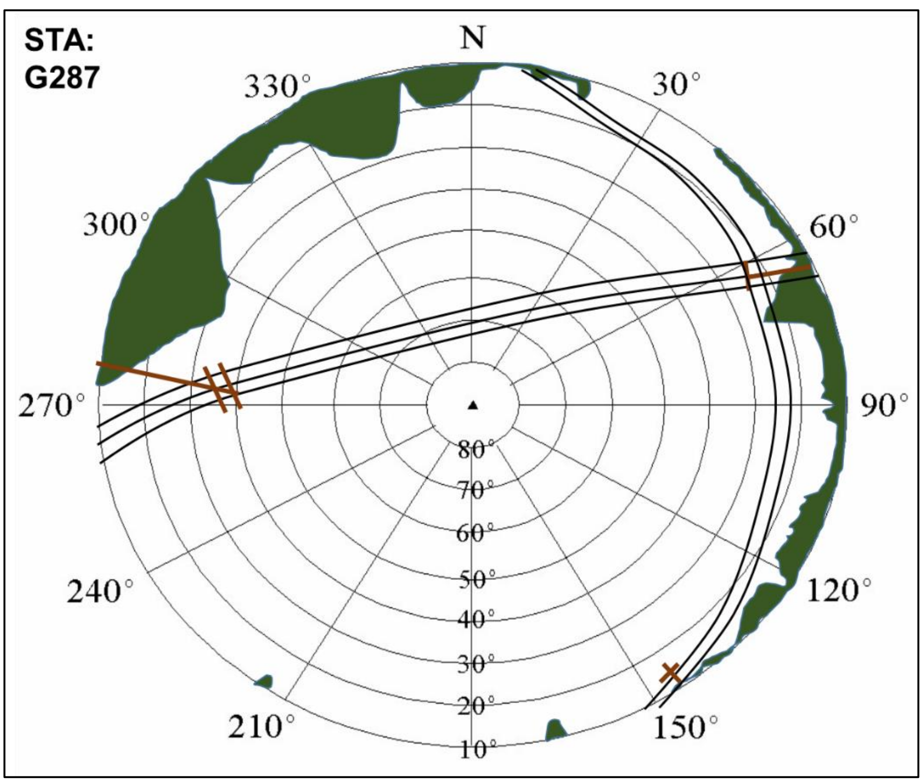

STA:
PEAK
N
Small tree 12' away
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°

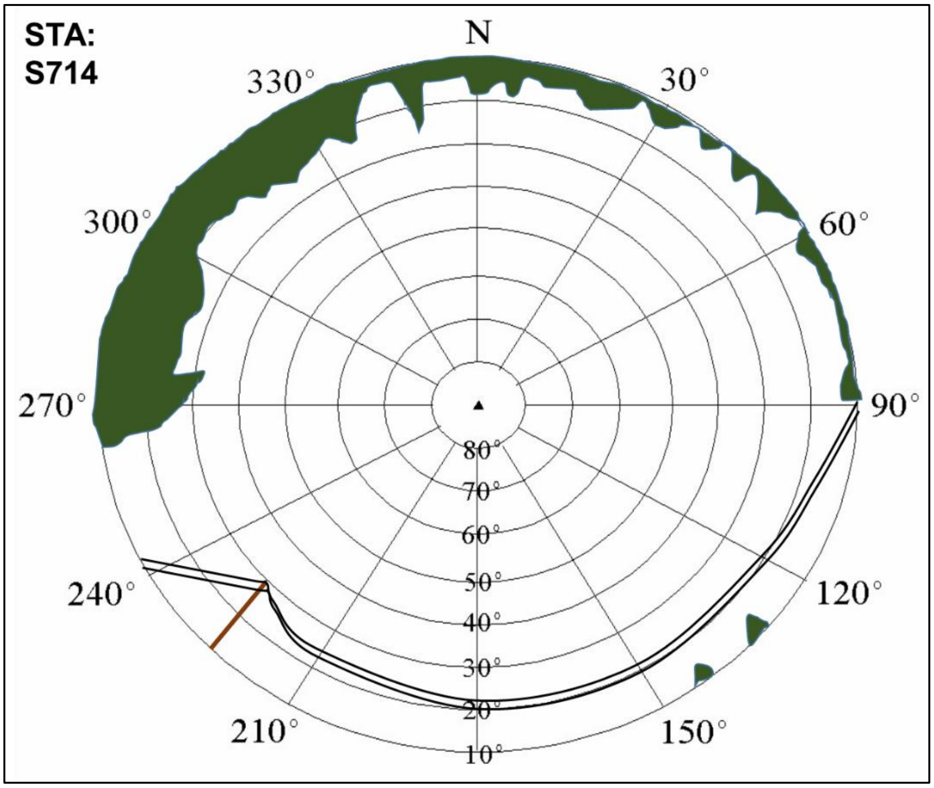

STA:
G728
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°

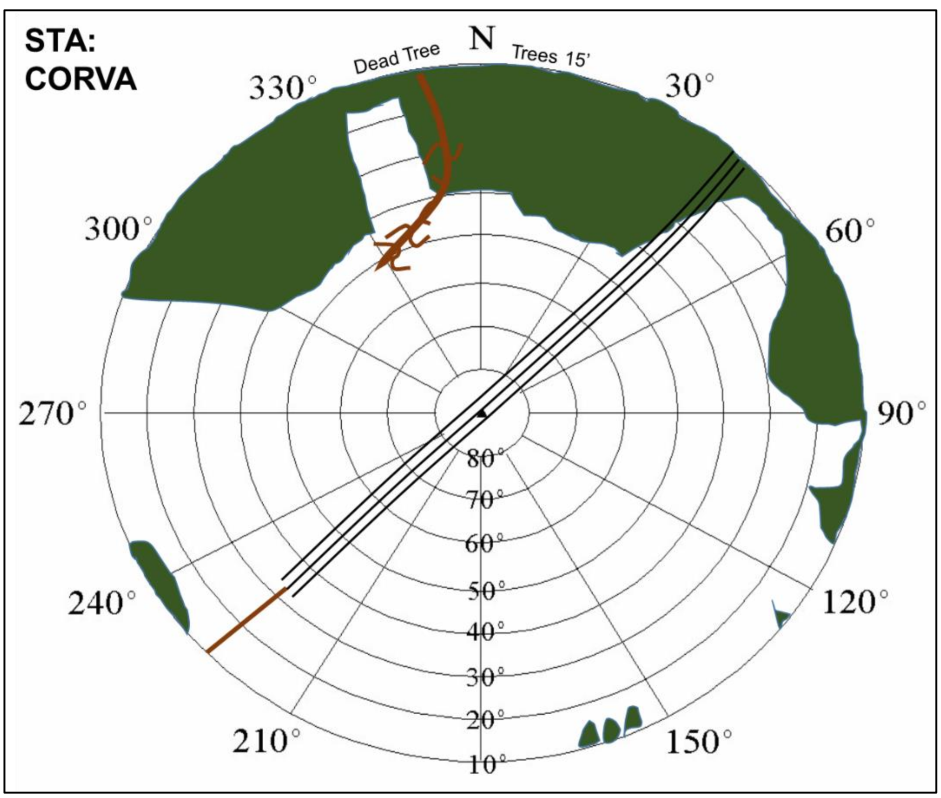

STA:
NESMITH
N
Power pole 60'
Steel Sign 55'
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°
Power pole 75'

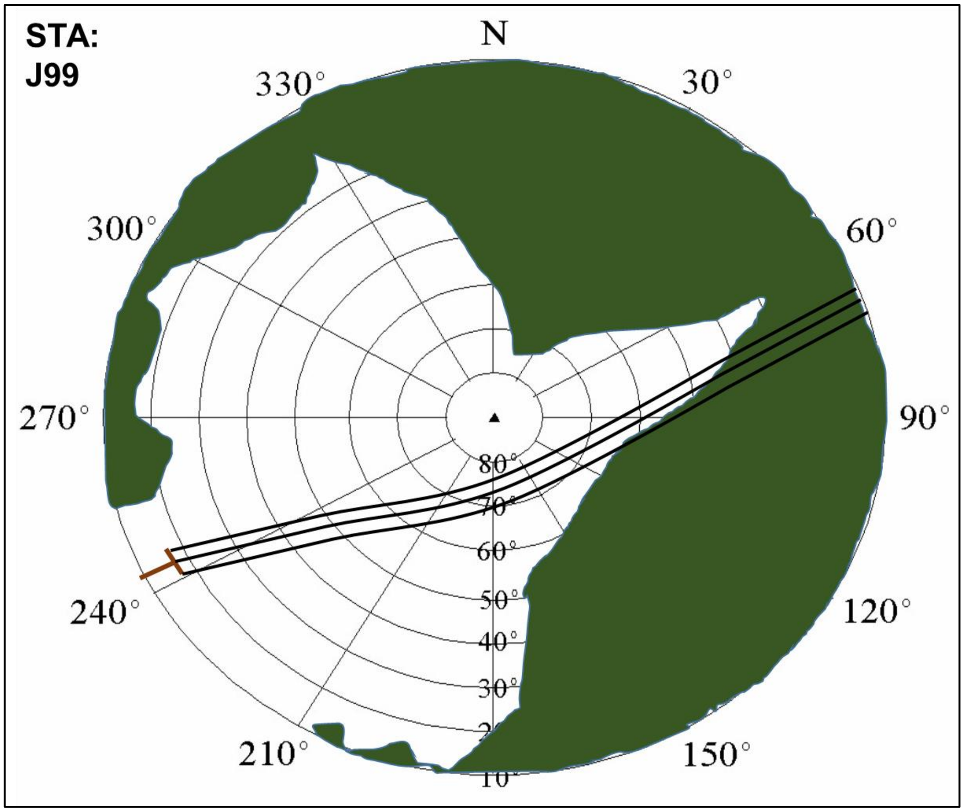

STA:
PEAV
N
330°
30°
Building
300°
60°
270°
90°
80°
70°
60°
240°
120°
Maple Tree 25' away
210°
150°
10°


STA:
Z714
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°


STA:
G287
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°


STA:
S714
N
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°


STA:
CORVA
Dead Tree
N
Trees 15'
330°
30°
300°
60°
270°
90°
240°
120°
210°
150°
80°
70°
60°
50°
40°
30°
20°
10°


STA:
J99
N
330°
30°
300°
60°
270°
90°
80°
70°
60°
50°
40°
30°
240°
120°
210°
150°
10°