8. PRIMARY PROJECT CONTROL
8.1 INTRODUCTION

The purpose of this chapter is to specify the minimum standards and procedures for establishing Primary Project Control on NYSDOT capital projects.

Primary Project Control may pre-exist in proximity to a project site. The Regional Land Surveyor shall be consulted prior to conducting a primary project control survey to determine whether pre-existing control shall be used.

When the term primary project control is used in this manual, it refers to the azimuth pairs, GPS reference (base) stations, and bench marks (Figure III) established in proximity to the project site, and beyond the project limits. The primary project control is used to establish secondary project control within the project area, to control the engineering and real property acquisition work required for a project. Points established under these standards and procedures are in proximity to the work limits of the project and are considered to be permanent control for the project. In this manual, primary project control is not intended to include densification or extension of existing geodetic control networks.

Depending upon the project needs, primary project control may or may not be needed. It is possible that the NYSNet CORS/RTN may be suitable for all the positioning needs for a project. The regional land surveyor shall be consulted to determine whether or not primary project control is needed.

FIGURE III. NYSDOT Survey Control Classifications

![Diagram showing NYSDOT Survey Control Classifications]

- Horizontal Control Network
- Vertical Control Network
- Primary Project Control
- Secondary Project Control
- <2 Miles (<3k)
- <6 Miles (<10k)
- <6 Miles (<10k)
- 30-60 Miles (50 – 100k)

Not to Scale
PRIMARY PROJECT CONTROL

Survey procedures may differ depending upon the project type. Network design can affect the local accuracy of these control stations. See 8.4.1.3 Network Design for guidance on Network design by project type. These guidelines are designed to ensure that a survey performed with GPS technology is repeatable, legally defensible, and referenced to the NSRS, by demonstrating the following:

1. Elimination or reduction of known and potential systematic error sources.
2. Occupational (station) and observational (baseline) redundancy to clearly demonstrate compliance with the stated accuracy.
3. Baseline processing, data adjustment and data analysis, to clearly demonstrate compliance with the stated accuracy.
4. Documented verification of the results to clearly demonstrate compliance with the stated accuracy.

GPS survey guidelines continually evolve with the advancements in equipment and techniques. Changes to these guidelines are expected as these advancements occur. The size, scope and site conditions of a project may also require variations from these guidelines. Any variations from these guidelines should be designed to meet the above criteria, and to achieve the accuracy standard of the survey as required by this document. All variations shall be discussed with the Regional Land Surveyor, and documented in the GPS Survey Report.

A network layout plan for the primary project control survey shall be submitted to the Regional Land Surveyor for approval, prior to the beginning of any field work.
PRIMARY PROJECT CONTROL

8.2 MONUMENTATION

Monumentation for primary project control should be a NGS 3-D rod, disk in massive concrete structure or bedrock, a poured in place concrete monument, or a drive-in type with expanding prongs (e.g., Berntsen FENO).

All primary project horizontal control stations shall be tied to 3 or 4 physical features, depending on the situation. Measure angles and distances, to tie objects, with a total station and record in the data collector. Observations to tie objects are normally recorded as a horizontal angle, slope distance, and zenith angle observation in the data collector. Distances can also be measured with a steel tape and entered into the data collector. The intent is for the ties to be plotted accurately and automatically in the digital graphics file with the rest of the survey information. Horizontal distances from tie objects should appear in reports, drawings, or computer files, unless otherwise noted.

Industrial Code Rule 53 of the New York State Department of Labor makes it the responsibility of the organization or firm actually setting monuments to contact the Underground Facilities Protective Organization (UFPO) in your area before beginning work. This requirement does not cover surveying per se, but specifically any mark or monument setting activity. Make reference to monument installation when calling UFPO at: (800) 962-7962 upstate; (800) 272-4480 in NYC and on Long Island.

8.3 MINIMUM STANDARDS

8.3.1 Primary Horizontal Project Control Standards

Primary Horizontal Project Control shall be established using GPS Relative Positioning Techniques and, depending upon the project requirements, may consist of:

8.3.1.1 Azimuth Pairs

Azimuth pairs consist of a pair of intervisible survey monuments, established to control a Total Station Positioning System (TPS) Survey (Secondary Project Control). The monuments in each pair shall be spaced at more than 1000 ft. (300 m) apart in order to meet azimuth reference requirements. Azimuth Pairs shall be set at a maximum of 2 mile (3 km) intervals throughout the project site, and suitably placed at locations to limit GPS obstructions, and to optimize their use at the project site.

All GPS surveys for primary horizontal project control azimuth pairs shall meet the standards of at least order C2-1 as defined in GEOMETRIC GEODETIC ACCURACY STANDARDS AND SPECIFICATIONS FOR USING GPS RELATIVE POSITIONING TECHNIQUES, Federal Geodetic Control Committee (FGCC), August, 1989. http://www.ngs.noaa.gov/FGCS/tech_pub/GeomGeod.pdf
PRIMARY PROJECT CONTROL

8.3.1.2 Base Stations

GPS Base stations, set within 6 miles (10 km) of the project site, suitably placed at locations to limit obstructions, to control secondary project control surveys, photogrammetric ground control surveys, and terrain data surveys.

All GPS surveys for primary horizontal project control base stations shall meet the standards of at least order C1 as defined in GEOMETRIC GEODETIC ACCURACY STANDARDS AND SPECIFICATIONS FOR USING GPS RELATIVE POSITIONING TECHNIQUES, Federal Geodetic Control Committee (FGCC), August, 1989. http://www.ngs.noaa.gov/FGCS/tech_pub/GeomGeod.pdf

8.3.2 Primary Vertical Project Control Standards

Primary Vertical Project Control shall be established using GPS, trigonometric or differential leveling techniques.

To qualify for adjustment, level line (differential, trigonometric or GPS) error shall not exceed .03 ft $\sqrt{D}$ where D is the length of the level line in miles (shall not exceed 8mm $\sqrt{D}$ where D is equal to the length of the level line in kilometers).

8.4 PROCEDURES

8.4.1 Primary Horizontal Project Control Procedures

8.4.1.1 Equipment

Surveyors shall employ dual frequency, full wavelength GPS receivers. No less than two (2) GPS receivers observing simultaneously shall be employed on the project control, while CORS are also included in the survey. Additional receivers could be employed on the project control to achieve increased productivity.

The antenna make and model employed on NYSDOT surveys must have been calibrated by NGS and have a defined phase center variability model for use in processing. Antennae types shall be accounted for in the GPS processing so that the correct phase center offsets are applied to determine the Antennae Reference Point (ARP).

8.4.1.2 Techniques

Static GPS observation techniques shall be used.
Post-processing of simultaneous field observations shall be used.
8.4.1.3 Network Design

GPS surveys for Primary Horizontal Project Control should be configured as geometrically closed networks of redundant vectors. All primary project control stations shall have two independent occupations. All primary project control azimuth pairs, and any other relatively short lines, must have two independent baseline vectors. Redundant baseline vectors are accomplished only by completely independent occupations of stations. Independent occupations require that the tripod or stand be reset and replumbed between sessions. So-called spaghetti, radial, or traverse survey schemes are not to be used for establishing primary project control on NYSDOT capital projects.

Redundancy in network design is achieved by connecting each network station with at least two independent baselines, including all stations in a series of interconnecting, closed loops, and repeating baseline vectors.

Closing the network geometrically means designing your network using good, closed geometry. A GPS or terrestrial network consists of a series of observations (baseline vectors) between network points. These baselines create closed, geometric figures such as triangles.

Structure your network design such that closed figures (loops) are created from baselines observed in two or more sessions. Each loop shall have at least one baseline in common with another loop. All new stations shall be included in a loop of not more than 10 baselines and a loop length of less than 60 miles (100 km).

Only independent baselines should be processed and incorporated into loop closures and included in the network adjustment. The use of independent baselines ensures that all data is used only once. Dependent data can introduce a bias into a data set and give erroneous results. If a GPS session uses 2 GPS receivers on project control and 2 CORS (a total of 4 receivers), the raw data produces 6 potential baselines:

\[
\text{# of Baselines} = \frac{N(N-1)}{2} \\
\text{# of Independent Baselines} = N - 1 \\
N = \text{# of receivers}
\]

Therefore only three independent baselines are created using a total of 4 receivers. The remaining three, observed at the same time, would be dependent, connected by common geometry and errors. Independent baselines must not form a closed figure. The surveyor decides which three baselines to process in order to achieve the network.

Azimuth pair baselines must be repeated to provide redundant observations in order to isolate and identify errors, and to provide network quality control checks, thereby increasing confidence in the results.
PRIMARY PROJECT CONTROL

Three Continuously Operating Reference Stations (CORS) that are included in the NYS Spatial Reference Network (NYSNet) shall be incorporated into the network design. Where possible, the three CORS shall surround the project site and be located in three different quadrants. In areas of the state where this is not possible, a National CORS or a NYHARN station may be included in the network in order to achieve network geometry surrounding the project site and located in three quadrants. Use of cooperative CORS that are not included in NYSNet should be avoided.

When planning network connections to CORS, the data availability and time series plots should be considered. When making connections to the NYHARN, the network accuracy of the station should be considered in the network adjustment.

The antenna type at the CORS station shall be identified in the processing. NGS phase center offset models shall be used to determine the offset from the ARP (Antenna Reference Point) to the antennae phase center. The coordinate for the ARP of the CORS station shall be used in processing.

Network statistics for Azimuth pair stations shall meet the requirements for order C2-I; network statistics for base stations shall meet the requirements for order C1, as defined in GEOMETRIC GEODETIC ACCURACY STANDARDS AND SPECIFICATIONS FOR USING GPS RELATIVE POSITIONING TECHNIQUES, Federal Geodetic Control Subcommittee, August, 1989. http://www.ngs.noaa.gov/FGCS/tech_pub/GeomGeod.pdf

Typical Network diagrams depend upon project type. For projects that will require Survey Terrain Data Collection using Total Station Positioning System (TPS) techniques only, the network diagram shown in Fig. IV is typical. This network establishes Azimuth Pair Control Stations only. If it is determined best to establish primary vertical control using GPS techniques, refer to Fig. VII in Section 8.4.2.3.
Line symbology indicates independent GPS baselines by session using 2 GPS receivers on project control, and 2 CORS (total of 4 receivers per session).
PRIMARY PROJECT CONTROL

GPS Base Stations should be established for projects that will require a photogrammetric control survey by Fast Static GPS or Local RTK (LRTK), or for projects that will make use of GPS techniques for terrain data collection, construction stakeout, or automated machine guidance by Fast Static GPS or Local RTK (LRTK) methods.

Base Stations should be established in a network separate from the Azimuth Pairs as shown in Figure V. Base Stations shall be within 6 miles (10 km) of the project site and should be about half the project length away from project centerline, surrounding the project site, in at least 3 quadrants. A fourth Base Station can be added to better surround the project site. For larger projects establish additional Base Stations to limit the distance to photogrammetric control targets or secondary project control points to no more than 6 miles (10 km). Azimuth Pair Stations should then be established as shown in Figure VI. Field work for these networks can be accomplished at the same time but the networks should be processed in this order.

If vertical control is required, Base Stations could be NSRS bench marks or eccentric stations tied to NSRS bench marks. Location of photogrammetric control targets using GPS should be configured as shown in Section 10.3.1.1.

If Network RTK (NRTK) is going to be used to conduct the photogrammetric control survey, there may be no need to establish local base stations. However, there may still be a need to tie to local NAVD88 Elevations surrounding the project site.

If NRTK is going to be used to establish Azimuth Pairs on a project, they may be surveyed as shown in figure IV.
FIGURE V. Horizontal Primary Control Survey To Base Stations

Line symbology indicates independent GPS baselines by session using 2 GPS receivers on the project control and 2 CORS (a total of 4 receivers per session).

Not to Scale
For projects that establish Base Stations, the Base Stations should be used as fixed control to establish Azimuth Pair Stations. The vectors used for the adjustment between the Base Stations (see Figure V) should be used for loop closures and adjustment of the Azimuth Control Stations. Figure VI is an example of the network design from the Base Stations to the Azimuth Stations for a project where two sets of Azimuth Stations were required. If vertical control is required, Base Stations could be NSRS bench marks or eccentric stations tied to NSRS bench marks.

FIGURE VI. Horizontal Primary Control Survey From Base Stations to Azimuth Pair Stations

Line symbology indicates independent GPS baselines by session using 2 receivers on the project.
PRIMARY PROJECT CONTROL

A GPS Plan for the Primary Project Control shall be submitted to the Regional Land Surveyor for review and approval, prior to beginning of any fieldwork. The GPS Plan shall include:

1. A network diagram showing independent baselines by sessions (indicate sessions by either (a) color, (b) line symbology, or (c) label/number the lines).
2. A summary of network statistics (see Section 8.7.2.1).
3. A session schedule.

Planning

Proper planning and network design shall be used in GPS surveys for primary project control.

Satellite almanacs used for observation planning shall be no more than 30 days old.

Stations should be situated in locations that minimize obstructions. In general, a clear view of the sky above 20 degrees is desired.

Field reconnaissance and pre-mission observation planning will be accomplished for all surveys. Sky visibility diagrams will be made for all stations. Diagrams will show magnetic azimuth and elevation angle to all obstructions above 15° to whole degrees. Obstruction data will be entered into a GPS observation planning program and multi-receiver sessions will be analyzed for acceptable observing times. Magnetic Declination should be compensated for in the planning program obstruction diagrams. Analysis should consider the number of available satellites and PDOP.

At least four healthy satellites shall be observed in common at all simultaneously occupied stations.

The PDOP should not exceed 6 during any GPS survey observations.

Static GPS observations for primary project control, using dual frequency receivers, the minimum observation session length shall be 30 minutes. For lines over 12 miles (20 km) add 2 minutes per mile (1 minute per 1 km) of GPS baseline length beyond 12 miles (20 km). A GPS baseline of **12 miles (20 km)** in length would require a minimum observation of 30 minutes. A GPS baseline of **25 miles (40 km)** in length would require a minimum observation of 50 minutes. Final choice of observation session lengths shall also consider site obstructions, predicted PDOP, number of satellites available, network configuration, and manufacturer guidelines.

<table>
<thead>
<tr>
<th>Baseline length</th>
<th>Minimum Observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;10 miles &lt;20 km</td>
<td>30 minutes</td>
</tr>
<tr>
<td>10-25 miles 20-40 km</td>
<td>45 minutes</td>
</tr>
<tr>
<td>25-30 miles 40-50 km</td>
<td>1 hour</td>
</tr>
<tr>
<td>30-50 miles 50-80 km</td>
<td>90 minutes</td>
</tr>
</tbody>
</table>
PRIMARY PROJECT CONTROL

Field Observation

GPS antennas should be set up over the points using fixed-height antenna tripods.

GPS observations will be collected at a minimum of 15 second data epochs. The tracking elevation mask angle should be $10^\circ$.

Vehicles should be parked away from or below the GPS antenna to minimize the chances of causing multipath signals.

Great care shall be taken in measuring and recording antenna heights. When using standard tripods, the antenna slope-height will be measured multiple times (per manufacturer's directions) and the average recorded.

8.4.2 Primary Vertical Project Control Procedures

Primary Vertical Project Control shall be established using GPS, trigonometric or differential leveling techniques.

Level runs shall begin and end on NSRS NAVD88 bench marks of first or second order accuracy classification. When first or second order bench marks are unavailable, the Regional Land Surveyor should be consulted for guidance. To confirm the integrity of the NSRS bench marks used, the leveling shall start on one NSRS bench mark and close on another NSRS bench mark. If leveling to a second NSRS bench mark is not practical or economical the Regional Land Surveyor should be consulted for guidance. When using GPS techniques, at least 4 well distributed (in four quadrants surrounding the project site) NSRS NAVD88 bench marks shall be incorporated into the GPS network.

When bench marks need to be established on the project site the bench marks should be incorporated into a primary differential level-run.

8.4.2.1 Differential Leveling

Digital levels with bar code rods shall be used. Use a sectional composite or invar bar-code staff.

Expend reasonable effort when balancing backsights and foresights. Use subsequent set-ups to make up for deficiencies in balancing. Difference in forward and backward sight lengths should never exceed $30 \text{ ft. (10 meters) per setup or 30 ft. (10 meters) per section.}$ Maximum sight length should not exceed $230 \text{ ft. (70 meters)}$ Minimum ground clearance of line of sight should be $1.6 \text{ ft. (0.5 meters)}$.

To qualify for adjustment, level line error, expressed in feet, shall not exceed $.03 \sqrt{D}$ where $D$ is the length of the level line in miles (shall not exceed $8\text{mm} \sqrt{D}$ where $D$ is equal to the length of the level line in kilometers).

8.4.2.2 Trigonometric Leveling
The total station used for trigonometric leveling should be a one-second least-reading instrument. The total station used should have a minimum DIN accuracy of two seconds (angle) and 2mm +2ppm (distance).

Account for the correct prism offset in all distance measurements.

Always use a target device with a retro-reflecter prism. The reflector alone does not make an adequate vertical target. The target/reflectors shall be tripod mounted.

Limit sight distances to about 1000 ft. (300 meters) to minimize the impact of small pointing errors. Longer sight distances may require additional pointings, resulting in a larger sample from which to derive a mean vertical angle.

Take great care in measuring the heights of instruments and targets for each set-up. Make redundant measurements to eliminate the chances of blunders. Make and record measurements at both the beginning and end of each occupation to check for settling, slipping, or misreading. When instruments and targets are exchanged on a tribrach, the HI or HT should be measured again. Never assume that they are the same or that the tripod hasn't deflected differently.

Make reciprocal observations (observations from each end of a line) on all observed lines. Observe each zenith angle with at least two D&R sets. The maximum spread of the sum of the direct and reverse zenith angles of a single set (one D&R) should be within 10 seconds. Keep re-observing rejected sets until two sets meet this tolerance.

The term reciprocal observations actually denotes zenith angles measured simultaneously from each end of a line. However, this procedure is rarely practical for highway surveying. In this case, zenith angle observations from each end of a line shall be separated by only a minimum amount of time in order to minimize any difference in the atmospheric conditions between stations during the observations. There are instances in which the atmospheric conditions are nearly identical during the times the zenith angles are observed at each end of the line. In these cases the effects of curvature and refraction on the zenith angle observations cancel out. However, there are other instances in which atmospheric conditions change significantly between the occupation of one point and the next. For best results in these instances repeat all observations.

Reciprocal observations correct distances and zenith angles for the effects of earth curvature and refraction. Automatic curvature and refraction corrections, applied in the instrument settings, usually apply corrections to horizontal distances and vertical changes only and do not affect the slope distances and zenith angles that are collected as part of a reciprocal observation. Therefore, automatic curvature and refraction corrections set at the instrument will have no effect on the reciprocal observation. When a line is observed from only one end, curvature and refraction corrections for determining vertical differences should be applied during processing.

Always try to occupy the known (initial and final) benchmark marks and all new bench marks in a trigonometric level run. Reciprocal observations cannot be made to and from a point that is not occupied. If a bench mark cannot be occupied, set up the total station
PRIMARY PROJECT CONTROL

within **70 ft. (20 meters)** of it and observe (in direct and reverse) the foresight vertical difference only. The very short distance will minimize the atmospheric effects.
Surround project site with a minimum of 4 well distributed NSRS bench marks within 6 miles (10 km) of the project site. NSRS bench marks should be distributed in all four quadrants of the network. Any NSRS Vertical Control Station of 2nd-Order or better, within 3 miles (5 km) of the project site should be incorporated into the network. If a large change in elevation exists through the project site, bench marks should be occupied at the base and summit of these elevation changes. Bench marks should have an NSRS vertical accuracy classification of 2nd order or better. Incorporate NSRS bench marks into the Primary Horizontal Project Control Survey. Whenever possible bench marks should be directly observed in the GPS network, however eccentric points can be used if necessary. Level connections to eccentric points shall follow primary vertical project control standards and procedures and field notes shall be included in the GPS control report. Independently occupy 100% of NSRS bench marks and vertical control stations. Occupations should be repeated 27-33 hours after initial observations.

Follow Primary Horizontal Project Control recommended procedures for planning and field observations. VDOP should be considered in session planning for a vertical control GPS survey. VDOP should be less than 4.

These Vertical Control Stations could also be used as Base Stations for LRTK and for establishing Azimuth Pair Stations. For a Network Diagram to establish Azimuth Pair Stations from Base Stations, see Figure VI.
8.5 DATA PROCESSING

8.5.1 Horizontal

GPS survey observations may be post-processed with the broadcast ephemeris or a precise ephemeris (rapid or final).

Process baselines using a default elevation mask of 15 degrees. Alter elevation mask when necessary.

The quality of baselines shall be checked according to the software manufacturer’s guidelines. All baseline solution types should be fixed. A fixed solution is obtained when the processor is able to find a set of integer values for the ambiguity terms that are significantly better than all other possibilities.

The RMS (root mean square) should be less than 0.05 ft. (15 mm).

Loop closures should be analyzed.

For Base Stations, in any component (X,Y,Z), “maximum” misclosure, not to exceed 0.8 ft. (25 cm). In any component (X,Y,Z), “maximum” misclosure, in terms of loop length, not to exceed 12.5 ppm. In any component (X,Y,Z), “average” misclosure, in terms of loop length, not to exceed 8 ppm. Repeat Baselines should be analyzed. In any component (X,Y,Z), “maximum” not to exceed 10 ppm.

For Azimuth Pairs, in any component (X,Y,Z), “maximum” misclosure, not to exceed 1.0 ft. (30 cm). In any component (X,Y,Z), “maximum” misclosure, in terms of loop length, not to exceed 25 ppm. In any component (X,Y,Z), “average” misclosure, in terms of loop length, not to exceed 16 ppm. Repeat Baselines should be analyzed. In any component (X,Y,Z), “maximum” not to exceed 20 ppm.

Only independent baselines should be incorporated into network adjustment.

A minimal constrained adjustment should be performed holding one NSRS station fixed in X,Y, and Ellipsoid Height. Adjustment residuals shall be analyzed for outliers. Once internal network accuracy is proven, a fully constrained adjustment shall be performed. Adjustment residuals shall be analyzed. Outliers, if any, shall be identified, and coordinate differences shall be reported to the Regional Land Surveyor.

Expected setup errors shall be accounted for in the least squares adjustment program. Refer to Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques, FGCC, 1989, Appendix D, “Expected Minimum/Maximum Antenna Setup Errors”. For primary project control, an expected setup error of 0.01 ft (0.3 cm) should be used.

Careful analysis of the minimally-constrained and fully-constrained least squares adjustments is the key to completing a project which meets its intended objectives. The following statistics shall be evaluated for each adjustment performed on a given project:
PRIMARY PROJECT CONTROL

The network variance of unit weight (variance factor) and degrees of freedom, for all variance groups, shall be evaluated. A variance factor of less than 1.5 and approaching 1.0 is considered a conservative statistic for geodetic control surveying. High variance factors may indicate blunders in the network; in this case the baseline solutions should be analyzed.

Using unusually large scalars (above 5) of error estimates for resolving outliers is not an acceptable method of passing the statistical tests. This usually indicates blunders in the network and should only be used to analyze outliers.

The RMS, minimum and maximum values, and the standard deviation of the absolute observation residuals should be equal to or better than the desired Accuracy. The sign of residuals at a particular point or network loop may indicate an undetected systematic error or blunder in the observations which are being smeared across that portion of the network. Any common trends in observation residuals are cause for further examination.

The standardized residuals, computed from the absolute residuals divided by their respective propagated standard error, shall be compared against the Chi-square test and Tau Criterion. Standardized residuals exceeding the Tau value should be further investigated as possible outliers. Failure to pass the Chi-square test is an indication that some or all of the a-priori errors have been improperly modeled. These tests are also highly influenced by external network constraints. It is not uncommon for good-quality networks to fail Chi-square and isolated residuals exceeding the Tau Criterion are not necessarily cause to reject the adjustment. These are strict statistical tests which should be used as tools to arrive at a final network adjustment meeting the objectives of the project.

Any significant changes between the statistics from the minimally-constrained adjustment and the fully-constrained adjustment should be investigated.

Posteriori errors shall be computed at the 95 percent (2 sigma) confidence level for the adjusted station coordinates and for the relative positions for all adjacent station pairs. Error ellipses provide a useful evaluation of the station confidence. Nearly circular and uniformly small error ellipses are an indication of a well-conditioned network. Irregularly shaped or unusually large error ellipse indicate problems with the satellite constellation used, gross errors or a weakness in the network design.

Apparent position shifts shall be computed between those obtained from the minimally-constrained adjustment and known values at the external network control. This is used as a check for systematic errors prior to computing any appropriate orientation parameters for rotation and scale and assigning provisional accuracy classification(s) for the project.

Error ellipse computed from random error propagation from a least squares adjustment, where the survey measurements have the correct weights and where the horizontal and vertical datum values are weighted using one-sigma network accuracies of the existing control, are used to evaluate the accuracy of a project. Relative error ellipse values are used to compute error circle radii for Local Accuracy classification. The evaluation shall be made from each point to all adjacent points regardless of the direct connections in the
network. Point error ellipses are used to compute error circle radii for Network Accuracy classification.

After a satisfactory standard deviation of unit weight is achieved using realistic a-priori error estimates, a fully constrained adjustment can be performed.

The fully constrained adjustment fixes the coordinates of the known NSRS control stations, thereby adjusting the network to the datum of the control stations. A consistent control reference network (NSRS) shall be used for the constrained adjustment.

8.5.2 Vertical

8.5.2.1 Differential Leveling

Differential level lines should be adjusted proportionally, according to the number of turns between bench marks.

8.5.2.2 Trigonometric Leveling

Processing software used to reduce trigonometric leveling notes shall mean the change in elevation, forward and backward along the traverse legs, while accounting for the target heights at each observation.

Trigonometric level runs should be adjusted by apportioning the errors according to the distances of the legs.

8.5.2.3 GPS Leveling

Follow the same procedures as in Section 8.5.1 Horizontal, including the following requirements:

Use fixed height tripods for all observations.

Minimum observation session length of 30 minutes for baselines from bench marks to project control.

Use a precise ephemeris.

Fixed baseline solutions are required on all baselines.

RMS of processed baselines should be less than 0.05 ft. (1.5 cm).

The Processed GPS Baselines should be checked for quality according to the software manufacturer guidelines. Repeat baselines shall be checked for misclosure. In any component (X,Y,Z), “maximum” not to exceed 20 ppm. The ellipsoid height difference of all repeat baselines shall be 0.07 ft (2 cm) or less.

Loop closures should be checked for misclosure. In any component (X,Y,Z), “maximum” misclosure, not to exceed 1 ft (30 cm) in any component (X,Y,Z), “maximum”
misclosure, in terms of loop length, not to exceed 25 ppm. In any component (X,Y,Z), “average” misclosure, in terms of loop length, not to exceed 16 ppm.

Perform a minimally constrained adjustment. Constrain one latitude, one longitude and one orthometric height value. Using the results from the constrained adjustment detect and remove all outliers. These two steps shall be repeated until all outliers are removed.

Compute differences between the set of orthometric heights from the minimally constrained adjustment using the latest geoid model and the published NAVD88 bench marks.
Determine which bench marks have valid NAVD88 height values. Differences between valid bench marks need to agree within $0.07 \text{ ft.} (2 \text{ cm})$.

Perform a fully constrained adjustment fixing one latitude and one longitude and all valid NAVD88 height values. At least three bench marks shall be held fixed in a fully constrained adjustment.

After a satisfactory standard deviation of unit weight is achieved using realistic a-priori error estimates, a fully constrained adjustment can be performed.

The fully constrained adjustment fixes the coordinates of the known NSRS control stations, thereby adjusting the network to the datum of the control stations. A consistent control reference network (NSRS) shall be used for the constrained adjustment.

8.6 QUALITY CONTROL

The licensed land surveyor in charge of the survey shall be responsible for the integrity and quality control of all survey data produced.

8.7 REPORTING

8.7.1 Field Notes

8.7.1.1 Horizontal

Raw data files in the manufacturer's format, along with data files in RINEX format shall be submitted for primary horizontal control surveys in digital format. Also, baseline components in NGS g-file format shall be submitted.

8.7.1.2 Vertical

Raw Data files and processed files shall be submitted for primary vertical project control surveys.

8.7.2 Survey Reports

The primary project control is used to establish secondary project control within the project area, to control the engineering and real property acquisition work required for a project. The surveyor can use the survey report to demonstrate and document to the customer the integrity of the control survey. It can also be used to guide future users of the primary project control.

Use page separators with labeled tabs, and/or an electronic table of contents, to separate each section of the report for easy referencing.
PRIMARY PROJECT CONTROL

8.7.2.1 Horizontal (GPS Survey Report)

Reporting of Primary Horizontal Project Control should include and progress as follows:

Table of Contents, Introduction, Personnel (field and office, job titles and job function), licensed land surveyor's certification, equipment calibration records and reports,

Project site location map (showing the project location and the established primary control monuments).

Project Narrative stating goals of project, standards to be achieved, NSRS monuments/bench marks used, monuments/bench marks established, coordinate system/datum/zone, geoid model, equipment and software, etc. Describe the procedures used. Comments/Recommendations

Network Diagram showing network connections, independent baselines/vectors by session, session lengths. (should evolve from the initial GPS Plan)

Summary of network statistics: (should evolve from the initial GPS Plan)
  Observing sessions (total number)
  Receivers observing simultaneously (total number)
  Total Number of Stations
  Number of new stations

Station occupations
  Single occupations (no redundancy)
  Two or more occupations (number/percent)
  Three or more occupations (number/percent)

Baselines determined
  All (trivial and nontrivial)
  Independent (nontrivial)

Repeat baselines (N-S/E-W/percent of non trivial)

Loop closure analyses
  Valid loops formed? /Number of stations not included in a loop
  Loops containing baselines from (2 or more/3 or more) sessions.
  Loops formed

Geometric relative position classification (based on FGCC specs)

Observation schedule
Maximum PDOP during session
Minimum number of satellites during session

NGS Data Sheets for NSRS Stations (NGS datasheet format) (Include CORS Coordinate Sheets for National CORS)
Control Data Sheets for Primary Project Control
PRIMARY PROJECT CONTROL

GPS Obstruction diagram for primary project control stations

Summary of baseline statistics showing quality of baselines

<table>
<thead>
<tr>
<th>Solution Type</th>
<th>Reference Variance</th>
<th>Ratio</th>
</tr>
</thead>
</table>

Summary of Loop Misclosures


Summary of Fully Constrained Adjustment showing values of constrained coordinates and heights, and network reference factor. Relative error ellipse values, largest relative error ellipse value for stations established by the survey. Point error ellipse values, largest point error ellipse value.

Adjusted Coordinates reported in NYSPCS, shown to four decimal places, ellipsoid heights shown to the nearest hundredth of a foot (millimeter) and orthometric heights shown to the nearest hundredth of a foot. Accuracy statement for the survey, per FGCC Specifications. Combined Factor, Grid Scale Factor, and Ellipsoidal Reduction Factor.

8.7.2.2 Vertical

Differential and Trigonometric Leveling (Survey Report)

Reporting of Primary Vertical Project Control should include and progress as follows:

Table of Contents, Introduction, Personnel (field and office, job titles and job function), licensed land surveyor’s certification, equipment calibration records and reports,

Project site location map (showing the project location and the established primary control bench marks).

Project Narrative stating: goals of project, standards to be achieved, Primary bench marks used, secondary bench marks established, datum references, equipment and software, etc. Describe the procedures used. Comments/Recommendations.

Description of the procedures used.

A summary Table showing the Summary of misclosures, including:

<table>
<thead>
<tr>
<th>From/To</th>
<th>Benchrun Length (Miles)(Km)</th>
<th># of turning points</th>
<th>Record starting elevation</th>
</tr>
</thead>
</table>
PRIMARY PROJECT CONTROL

Record ending elevation
Calculated ending elevation
Calculated misclosure

Allowable Misclosure based upon .03 ft $\sqrt{D}$ where D is the length of the level line in miles (8mm $\sqrt{D}$ where D is equal to the length of the level line in kilometers).
PRIMARY PROJECT CONTROL

Network Diagram showing network connections.

Control Data Sheets for NSRS Bench marks

Control Data Sheets for the primary project control bench marks established (elevations shown to the nearest hundredth of a foot) (elevations shown to the nearest millimeter)

Summary of adjustment method and statistics

Adjusted elevations reported in NAVD 88 shown to nearest hundredth of a foot, (millimeter) and accuracy statement for the survey (per FGCC Specifications).

Vertical control map (based on a suitable scaled plan, e.g. 1:6000, showing the location of all secondary project control bench marks, relative to named cross streets and other prominent features).

GPS Leveling (GPS Survey Report)

See Section 8.7.2.1

8.8 REFERENCES

2. Federal Geodetic Control Committee’s (FGCC) publication “Standards and Specifications for Geodetic Control Networks” published September, 1984