OC SURVEY STANDARDS MANUAL

2020
Best Practices Guide

COUNTY OF ORANGE CALIFORNIA

OC Public Works

County of Orange

OC Public Works Department

Santa Ana, CA

Shane L. Silsby, Director

Prepared by OC Survey

Field Services Unit

Kevin Hills, County Surveyor

Orange County Board of Supervisors

Michelle Steel
Chair
County of Orange Supervisor Second District

Andrew Do
Vice Chair
County of Orange Supervisor First District

Donald P. Wagner
County of Orange Supervisor Third District

Doug Chaffee
County of Orange Supervisor Fourth District

Lisa A. Bartlett
County of Orange Supervisor Fifth District

OC SURVEY STANDARDS MANUAL

CHAPTER 1
STATIC GNSS

Policy Statement

Any survey which incorporates a Static GNSS network shall conform to the specifications as defined in this document.

General Statement

The term "Static GNSS" mentioned herein is used as a general term, and may also refer to procedures more accurately classified as "Rapid Static GNSS".

Accuracy Standards

The general standard to be applied to most projects is a combined (relative) positional accuracy of **1:10,000** (at a 95% confidence level, or 2 sigma), or a combined distance error of **≤ 0.033** feet for connection distances shorter than 330 feet. Relative positional accuracy is a measure of the accuracy of point positions in relation to each other, and is not to be confused with the measure of traverse closure expressed as a ratio. This accuracy standard applies to any survey which:

- Locates or establishes land boundaries, rights of way, or centerline alignments
- Establishes control for photogrammetric or topographic surveys
- Establishes first generation control for construction projects

Variations from the above standard are as follows:

- Projects which establish new primary control stations for the OC Survey Geodetic Network shall conform to a minimum combined (relative) positional accuracy of **1:20,000**. In addition, each new control station shall have a computed network accuracy (error ellipse semi-major axis) **≤ 0.033 feet**.
- Projects which incorporate CGPS stations to establish a pair of primary project control points which serve as the project basis of bearings shall conform to a minimum combined (relative) positional accuracy of **1:20,000**. In addition, each of these new control points shall have a computed network accuracy (error ellipse semi-major axis) **≤ 0.033** feet.
- In some rare instances, a project may require less stringent accuracy standards than those described above. This determination will be made by the Senior Land Surveyor on a project by project basis.

The procedure for determining whether or not these accuracy standards have been met is outlined below in the section entitled "Adjustment of the Network."

Legacy Control vs. CGPS

It is the policy of OC Survey that CGPS stations will be used as the primary basis for future control. Legacy control is generally only to be used when tying into or working adjacent to an older project, or where project location prevents selection of CGPS stations in at least three of four geographic quadrants. The decision to hold legacy control over CGPS stations must be made by the Senior Land Surveyor. In the case of work being conducted by a consultant, the decision will be made by the designated OC Survey Project Manager.

Legacy Control

Advantages:

Orange County has the luxury of access to an extensive network of legacy control stations, on a roughly half-mile grid across most of the County. Most projects can easily be encircled by at least 4 existing stations. As baselines are generally under a mile in length, occupation times can be cut to a minimum. An added bonus is that many of these stations represent a position of record, such as a centerline intersection, so use of the stations may provide essentially "free" boundary points.

Disadvantages:

Each of these stations must be occupied at least twice, adding time to the field survey and complexity to the mission planning. Many of the stations are located in busy street intersections, creating potential safety concerns. Perhaps most importantly, the original survey of these stations was performed over 20 years ago, and with time, subsidence, and shifting tectonic plates, the integrity of this network has degraded and will continue to degrade. Also, over the years, many of the original monuments have been destroyed and replaced by various surveyors. These monuments become R1 (or even R2) status, and the actual positions relative to the published coordinate values may be questionable.

Methodology:

The survey must be constrained to a minimum of 4 legacy control stations, with coordinates based on the same epoch date and published by OC Surveys. The control stations are selected so as to create a polygon which fully encompasses the project area.

While the network may be designed and adjusted using only GNSS vectors (stand-alone), combining conventional traverse (total station) data with GNSS vectors will result in a network with higher relative positional accuracy (and may eliminate the minimum spacing requirements that follow).

The minimum allowable spacing for points in stand-alone networks shall be dictated by the following criteria:

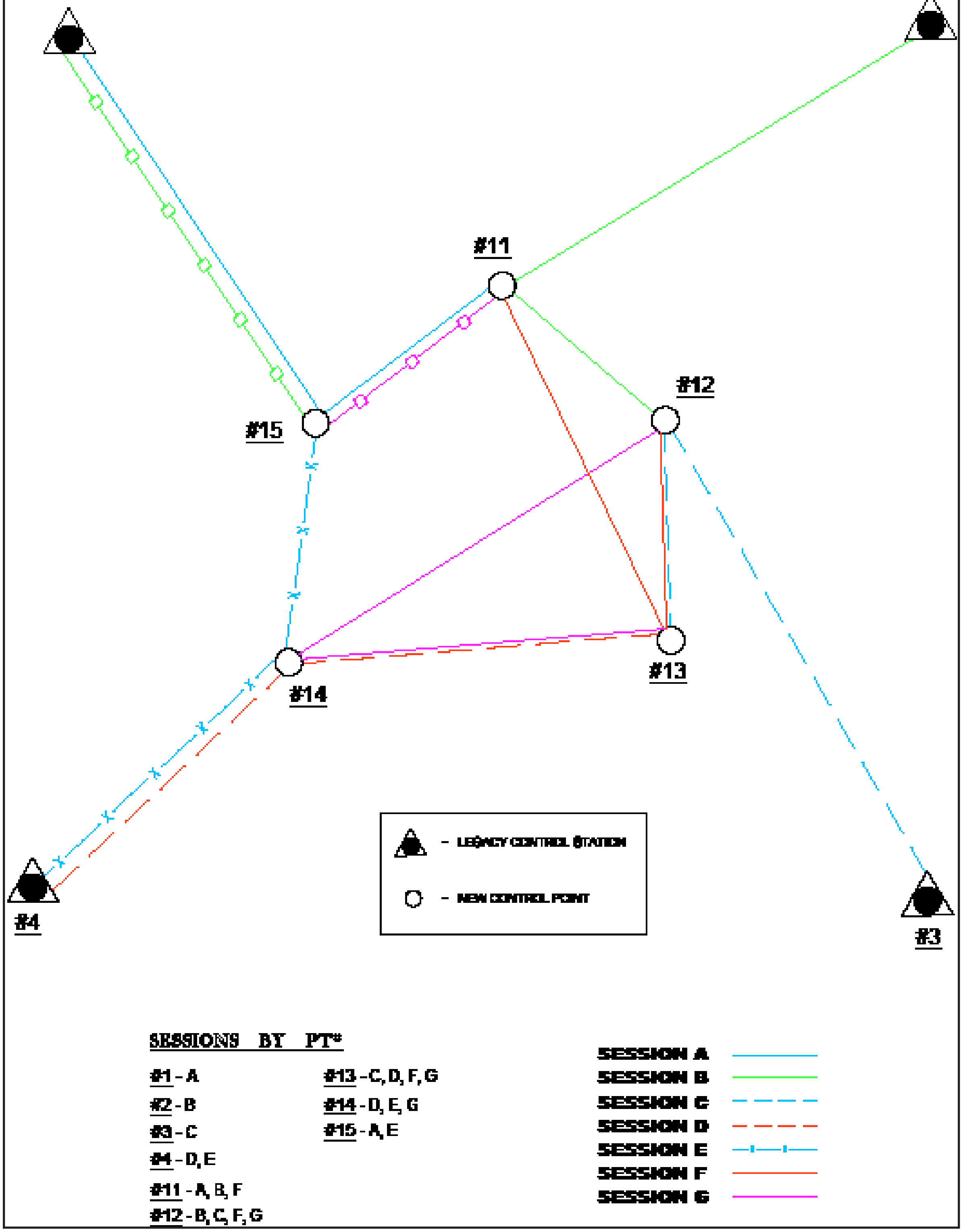
- Trimble R10 receivers, rated for static surveys at 3mm + 0.5 ppm at 68% confidence level (1 sigma): a minimum spacing of **300 feet** when tied to legacy control, at an average distance of 4,000 feet (see the formula shown in "Appendix A, Section 1")
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

Sessions should be planned with the intention of occupying each new control point at least twice and each existing control point at least once, while observing all desired baselines at least once. Careful planning yields increased efficiency and assurance that all desired baselines are observed.

Figure 1 represents an example of a static network with the following parameters:

- 4 existing legacy control stations
- 5 new control stations to be tied in
- 3 GNSS receivers
- Total of 7 occupation sessions
- 2 non-trivial baselines measured per session (only baselines shown in Figure 1 are to be processed)
- 15 minute sessions with approximate 15 minute move times
- All desired non-trivial baselines are captured in approximately 4 hours

FIGURE 1


#1
#2
#11
#12
#15
#13
#14
#4
#3
- LEGACY CONTROL STATION
- NEW CONTROL POINT
SESSIONS BY PT#
#1 - A
#2 - B
#3 - C
#4 - D, E
#11 - A, B, F
#12 - B, C, F, G
#13 - C, D, F, G
#14 - D, E, G
#15 - A, E
SESSION A
SESSION B
SESSION C
SESSION D
SESSION E
SESSION F
SESSION G

CGPS

Advantages:

The use of CGPS stations offers the luxury of 3 to 4 essentially "free" receivers, operating continuously. This saves time in the field and reduces the complexity of mission planning.

Disadvantages:

CGPS stations are often located miles from the project site and thus require longer occupation times. However, option "B" presented below introduces methodology which mitigates the need for longer occupations.

Methodology:

The survey must be tied to (but not necessarily constrained to) a minimum of 3 CGPS stations. The CGPS stations shall be selected so as to create a polygon which fully encompasses the project area.

While the network may be designed and adjusted using only GNSS vectors (stand-alone), combining conventional traverse (total station) data with GNSS vectors will result in a network with higher relative positional accuracy (and may eliminate the minimum spacing requirements that follow). The minimum allowable spacing for networks tied to CGPS stations in stand-alone networks shall be dictated by the following criteria (based on the Trimble R10 receiver specifications shown above and the formula shown in "Appendix A, Section 1"):

- a minimum spacing of **500 feet** when tied to CGPS stations at an average distance of 32,000 feet
- a minimum spacing of **300 feet** when tied to primary project control (see option "B", Figure 3b and Figure 3c below) at an average distance of 4,000 feet

Occupation sessions should be planned with the intention of measuring all desired baselines at least once, while occupying each control point at least twice. Careful planning yields increased efficiency and assurance that all desired baselines are measured.

Two of the possible methods for designing and processing a network tied to CGPS stations are detailed below:

A. Processing each new control point independently against the CGPS stations: One advantage to this method is that almost no mission planning is needed, and the survey can be performed with just one receiver. Data can be collected by one person (two if working within an active roadway), freeing other crew members to simultaneously perform additional tasks (running levels, etc.). One disadvantage of this method is that baselines are only being measured and processed from the CGPS stations; baselines from point to point within the project become trivial baselines. Without the addition of conventional traverse data, resultant relative positional accuracy may not meet standards as defined by this document. Another disadvantage is that because the CGPS stations are located so far apart, occupation times are increased dramatically.

Figure 2 represents an example of a static network with the following parameters:

- 3 CGPS control stations
- 4 new control points to be tied in
- 1 receiver used
- Total of 8 sessions (two sessions per point)
- 30 minute sessions with approximate 15 minute move times
- All desired baselines are captured in approximately 6 hours.

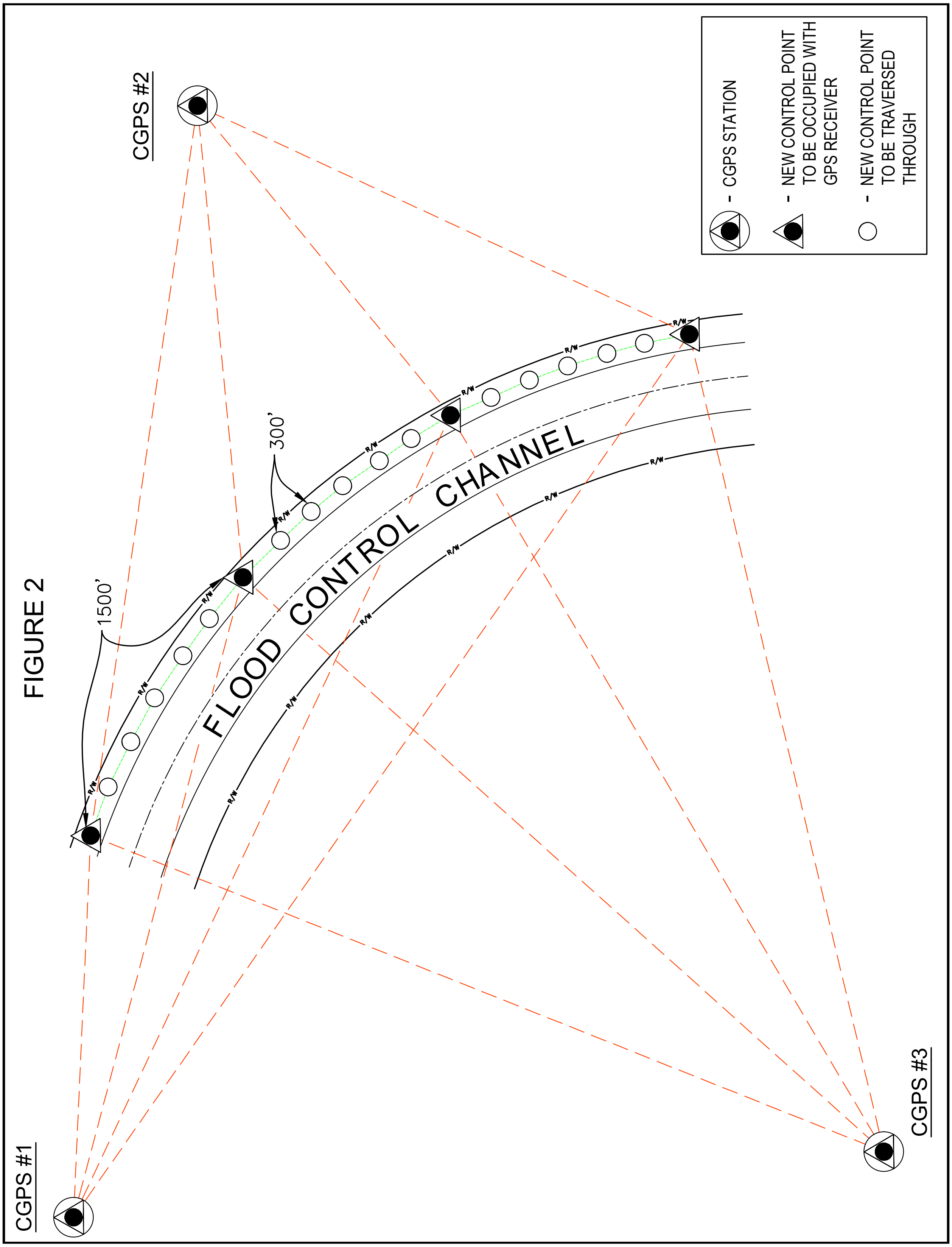


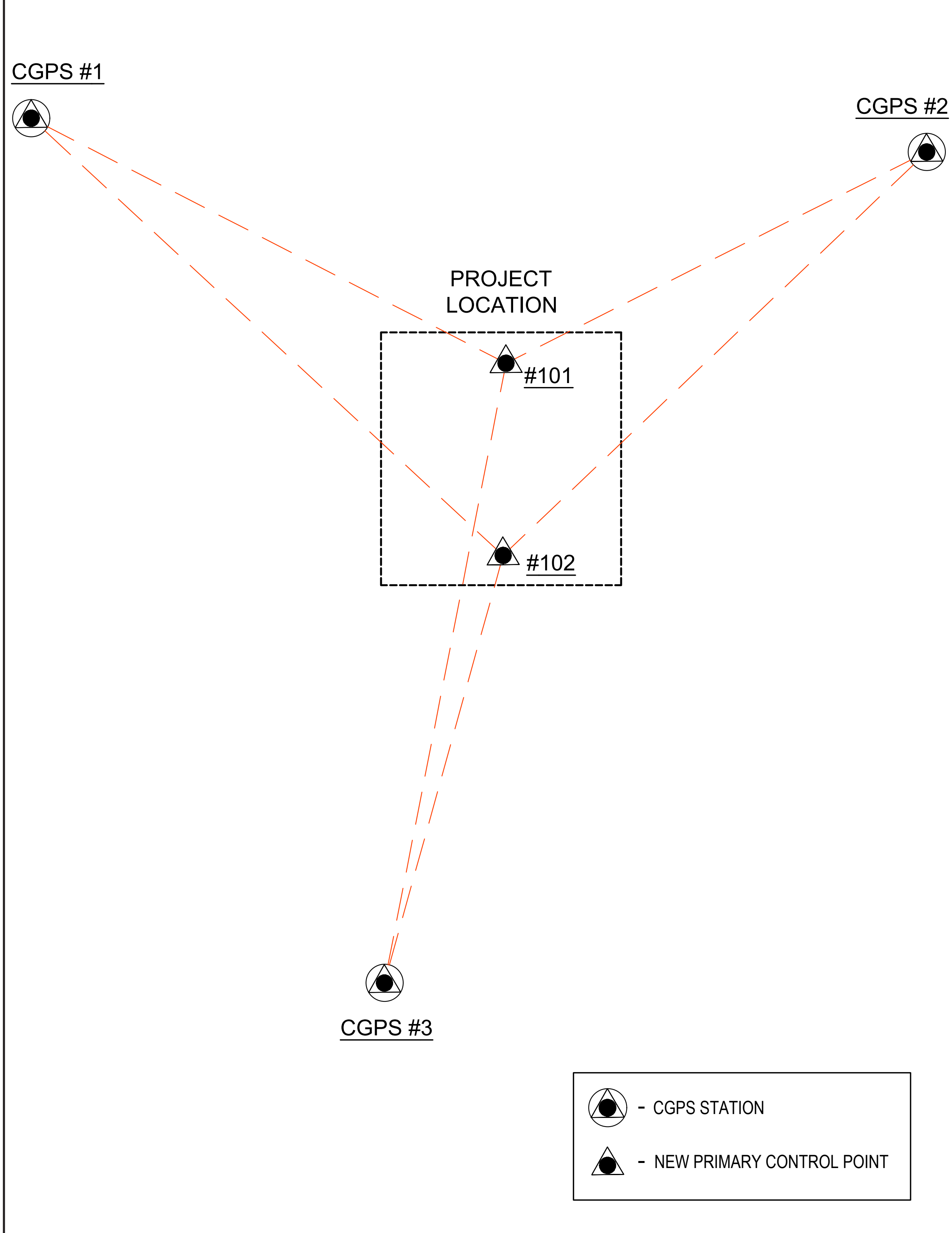
FIGURE 2
CGPS #1
CGPS #2
CGPS #3
1500'
300'
FLOOD CONTROL CHANNEL
R/W
- CGPS STATION
- NEW CONTROL POINT TO BE OCCUPIED WITH GPS RECEIVER
- NEW CONTROL POINT TO BE TRAVERSED THROUGH

B. Establish 2 primary project control points relative to the CGPS stations; establish secondary control points relative to these primary control points: This option employs three or more receivers and requires more personnel and more complex mission planning than Option "A" above, but will result in a network with a higher relative positional accuracy. The primary project control points are tied to CGPS stations with 2 independent occupations of ≥ 1 hour, with a minimum time differential (time of day) of 2 hours. These points are held to higher network and relative positional accuracy standards (**0.033 feet and 1:20,000, spaced a minimum of 1000 feet apart, as explained above**) and will become the project "basis of bearings". The secondary control points are tied to the primary control with occupation sessions of 10-15 minutes.

Figure 3-A and **Figure 3-B** represent an example of a static network with the following parameters:

- 3 CGPS control stations
- 2 new primary control points to be tied in
- 5 new secondary control points to be tied in
- 3 receivers used
- The first session lasts approximately 110 minutes, during which baselines are measured from the CGPS stations to each of the two primary control points. During this 110 minute session, a third receiver occupies each of the secondary control points for 10 minutes each.
- The next session is 15 minutes long, and measures the baseline between the two primary control points while the third receiver is moving.
- The final session is 110 minutes long and repeats the process from the first session and provides a second occupation of each secondary control point.
- 2 non-trivial baselines measured per session; 1 non-trivial baseline measured during the move session (only baselines shown in Figure 3a and 3b are to be processed).
- All desired baselines are captured in approximately 4 hours.

FIGURE 3-A


CGPS #1
CGPS #2
PROJECT
LOCATION
#101
#102
CGPS #3
- CGPS STATION
- NEW PRIMARY CONTROL POINT

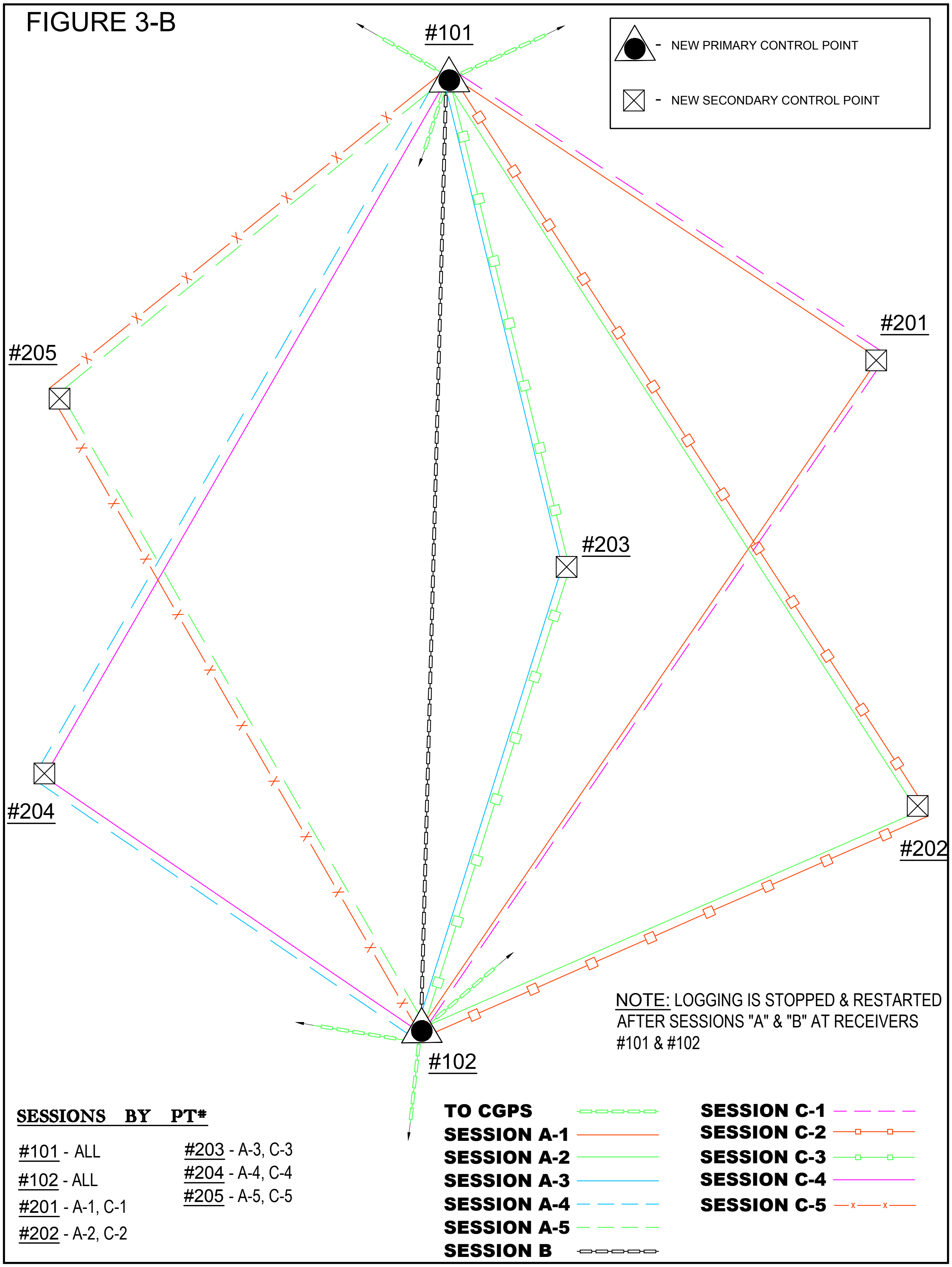


FIGURE 3-B
#101
NEW PRIMARY CONTROL POINT
NEW SECONDARY CONTROL POINT
#201
#205
#203
#204
#202
#102
NOTE: LOGGING IS STOPPED & RESTARTED AFTER SESSIONS "A" & "B" AT RECEIVERS #101 & #102
SESSIONS BY PT#
#101 - ALL
#102 - ALL
#201 - A-1, C-1
#202 - A-2, C-2
#203 - A-3, C-3
#204 - A-4, C-4
#205 - A-5, C-5
TO CGPS
SESSION A-1
SESSION A-2
SESSION A-3
SESSION A-4
SESSION A-5
SESSION B
SESSION C-1
SESSION C-2
SESSION C-3
SESSION C-4
SESSION C-5

Additional Field Procedures

- Conditions which may generate multipath or obstruct view of the satellites, such as overhead power lines, nearby trees, or adjacent buildings, should be avoided.
- Each existing control point shall be occupied at least once and each new control point shall be occupied at least twice (regardless of whether or not conventional traversing is incorporated into the survey), said occupations having a minimum time differential (time of day) of 1 hour. This time differential should be extended to 2 hours when establishing new primary control points.
- GNSS receivers shall be mounted on either a tripod/tribrach configuration, or a fixed height or locking-pin rod. This rod shall have three support legs and a center leg which freely turns 360 degrees. A standard layout rod with supporting bipod shall NOT be used for any static GNSS occupations.
- Receiver HI is measured two times, one measurement in feet and one in meters, and a unit conversion applied to verify the HI before the receiver is moved.
- Receivers remaining in place for consecutive sessions shall be re-levelled and re-centered between each session. HI measurements are repeated as well.
- Data is logged with an elevation mask of 10 degrees, but processed with a mask of 15 degrees.
- Data is logged at an interval of 15 seconds.
- While the receiver is logging data, a full description of the physical monument is recorded and a digital image is captured.
- Occupation session data is recorded on the relevant setup sheet ("Static GPS Set-Up Sheet – Loop Network" or "Static GPS Set-Up Sheet – Hub Network"). A separate set-up sheet is used for each receiver.
- Considerable care should be given to point naming conventions. Each time a point is occupied, it is to be given the same name. There is no need to use A, B, C etc. for subsequent occupations of the same point.

Monumentation

Monuments set as control points during the course of a GNSS survey shall meet the following criteria:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.

- Monuments which fall in non-paved areas shall consist of an iron pipe with a tag or disk, or a rebar with an aluminum cap. Rebar must be set a minimum of 3 inches below the ground surface.
- All tags/washers/disks/caps referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge, and shall also be stamped "CP" or "CONTROL POINT".
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe or rebar.

Adjustment of the Network

All GNSS data shall be adjusted by least squares adjustment software, in conformance with Chapter 12 – Network Processing. Note that although a network adjustment may be performed using only GNSS vectors (stand-alone), combining conventional traverse data with GNSS vectors will result in a network with higher relative positional accuracy.

Statistical analysis of the network adjustment shall be performed to ensure that a minimum combined (relative) positional accuracy of **1:10,000** (or **1:20,000** where required above) has been achieved for all connected monument pairs. Although this computation is automatically performed in most network adjustment software, the formula for this computation is shown in "Appendix A, Section 2."

Connections of very short distances often will not meet this **1:10,000** standard. An alternative standard for distances of less than 330 feet is shown in "Appendix A, Section 3."

In the event one or more pairs of monuments fail to pass these relative positional accuracy criteria, the network adjustment shall be reviewed and a determination made by the Senior Land Surveyor (or Project Manager) as to whether or not additional observations will made in order to improve geometry, increase redundancy, or further isolate errors.

Important Note:

Once a network has been adjusted and coordinates are reported to another entity (e.g.: Boundary Analysis Unit or Mapping Unit), these coordinates shall be deemed final. Should supplemental control or boundary ties be needed, the primary coordinates shall be fixed in subsequent adjustments. Only in the event that erroneous data is discovered will previously reported coordinate values be changed.

Additional Resources:

Chapter 6 of the Caltrans Survey Manual and CLSA/CSRC GNSS Surveying Standards and Specifications are valuable resources which should be consulted before planning a GNSS survey.

Appendix A - Formulas

1. Minimum spacing for new control points to be positioned using static GNSS can be computed using the following formula:

   $$D = 10{,}000 \times \sqrt{(2 \times \{ [(1.96)(a)]^2 + [(1.96)(b)]^2 + c^2 \})}$$

where:

- D = minimum spacing (in feet) between static occupation stations
- a = manufacturer's millimeter rating at a 68% confidence level, (converted to feet)
- b = manufacturer's ppm rating at a 68% confidence level, times the average distance (in feet) from legacy control stations, and divided by 1,000,000
- c = estimated receiver positioning error (rod plumb or tribrach errors), commonly estimated to be 0.007 feet
- 1.96 = the multiplier from a 68% confidence level (1 sigma) to a 95% confidence level (2 sigma)

2. All connected monument pairs shall pass the following mathematical test:

   $$D \div \sqrt{(x^2 + y^2)} \geq 10{,}000$$ (or $\geq$ 20,000 where required above)

where:

- D = distance (in feet) between the pair of monuments being examined
- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

3. Connections of very short distances often will not meet the **1:10,000** standard defined by the formula in Section 2 above. An alternative standard for distances of less than 330 feet follows:

   $$\sqrt{(x^2 + y^2)} \leq 0.033 \text{ feet}$$

where:

- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

CHAPTER 2
RTK GNSS

Policy Statement

Any survey which incorporates Real Time Kinematic GNSS (RTK) shall conform to the specifications as defined in this document.

General Notes

In performing an RTK survey, a GNSS receiver occupies a known Reference Station and remains stationary (**Base Station**), while another roving GNSS unit is moved from point to point (**Rover**). A Real Time Network (RTN) provides permanently mounted, continuously operating Base Stations, while a traditional Base/Rover configuration requires physical set-up of the Base Station. In either case, baselines are computed between the Base Station GNSS receiver and the Rover GNSS receiver, which then computes a real-time position relative to the Reference Station position.

Although uses of RTK are limited when higher accuracies are required, RTK is ideally suited to the following types of surveys:

- Setting of slope stakes, rough grade stakes, limits of grading stakes, etc. (see Chapter 3 – Construction Staking)
- Performing a topographic survey of a dirt area (see Chapter 6 – Topographic Surveys)
- Performing a scour study survey (see Chapter 8 – Scour Study Surveys)
- Performing a hydrographic survey (see Chapter 9 – Hydrographic Surveys)
- Searching for monuments (which will subsequently be tied-in using a higher-order methodology)
- Flagging or painting City/County boundary lines (to establish limits of resurfacing, etc.)
- Layout of conceptual alignments for visual inspection
- Positioning aerial targets

Limitations of RTK

Beyond the uses listed above, it is important to recognize that RTK is generally not to be used as a stand-alone measurement tool when performing a boundary or control survey. RTK is best used to bolster a control network, not define it. In order to ensure realization of the 1:10,000 combined (relative) positional accuracy standard required by these types of surveys, the network should be adjusted using RTK measurements together with conventional traverse data. RTK occupation points are selected in such a way as to maximize strength of figure, while leaving the bulk of the data to be captured by conventional traverse. The occupation scheme will be determined by the Party Chief.

The minimum recommended spacing for points in RTK surveys shall be dictated by the following criteria:

- Trimble R10 receivers, rated for RTK surveys at 8mm + 1 ppm at 68% confidence level (1 sigma): a minimum spacing of 1200 feet when tied to CRTN Stations at an average distance of 32,000 feet; a minimum spacing of 700 feet when tied to published legacy control or local project control in a Base-Rover configuration at an average distance of 4,000 (see the formula shown in "Appendix A, Section 1")
- Minimum spacing for GNSS receivers with RTK survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

CRTN vs. Base/Rover Configuration

There are two basic options when performing an RTK survey within Orange County: the California Real Time Network (CRTN) and a traditional Base/Rover configuration. Each method has advantages and disadvantages and both are currently acceptable for use, as directed within the bounds of this document. In most cases, conditions on the ground will make the decision an obvious one. This decision must occur before planning and design of the survey. Design of the network and occupation scheme will be determined by the Party Chief.

CRTN

Advantages:

Use of the CRTN offers the luxury of an essentially "free" continuously operating Base Station, thus a survey may be conducted with a single GNSS receiver. Stations within the CRTN are of a known integrity. There is no set-up time associated with the Base Station. Link between the Base and Rover is established via cellular modem and thus not limited to line of sight or by distance.

Disadvantages:

Due to the wide spacing of CRTN Stations, the nearest Station may be up to 10 miles from an individual project location, resulting in longer initialization times and more importantly, a significantly larger PPM error. Correct modeling of errors associated with atmospheric conditions is critical to achieving accurate results, and as the Rover moves farther from the Base Station, the modeling of these atmospheric errors becomes less certain. Some of the CRTN Stations rely on solar power and may become non-operational during periods of prolonged inclement weather.

A robust cellular connection is required to connect to the server which broadcasts corrections. Corrections broadcast from some CRTN Stations do not include data from GNSS satellites: working with fewer satellites reduces horizontal and vertical accuracy and may make it difficult to obtain and maintain initialization.

Methodology:

The survey must be constrained to a minimum of 4 existing control stations. These control stations are selected so as to create a polygon which fully encompasses the project area. If positions will be needed in real-time, the data collected at these control stations is used to compute a Site Calibration (see Appendix B for Site Calibration procedures). If positions are not needed in real-time, data is still collected as described in Appendix B but will be processed later (see Adjustment of the Network, below).

After a link is established between Base and Rover and data is collected at the existing control stations, collection of topographic survey data or establishment of new control points may begin.

Each new control point shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 2 hours. These time differentials are required in order to ensure significantly different satellite geometry. Each of these occupations shall consist of either one measurement of 180 epochs, or three sequential measurements of 60 epochs each.

Each topographic data point shall consist of a single measurement of at least 5 epochs.

Additional Notes:

All survey data collected for a project shall be tied to the same CRTN Base Station and epoch.

Base/Rover Configuration

Advantages:

The Base/Rover configuration can be used in areas where no cellular service exists. Corrections broadcast from the Base Station include data from GNSS satellites, thus constellations of 12 or more satellites are not uncommon. This is especially beneficial for 3D surveys. The Base and Rover are rarely more than 1 mile apart, resulting in insignificant PPM errors. Orientation of the Base Station on an existing project control point may eliminate the need for Site Calibration.

Disadvantages:

The Base/Rover configuration requires utilization of an additional GNSS receiver and potentially an additional crew member. Care must be taken to mitigate the theft risk created when this stationary receiver is located in an area of public access. Additional set-up and break-down time must be factored into the daily work flow. Link between the Base and Rover is established via radio signal and is limited to line of sight, at a maximum range of about 1 mile.

Methodology:

Two of the possible methods for conducting a conventional Base/Rover RTK survey are detailed below:

A. Placing the Base receiver on a published legacy control point:

The legacy control point must be of the same epoch date as the current project.

This method eliminates the need for Site Calibration.

After a link is established between Base and Rover, a second published legacy point is checked and stored as described above, and collection of topographic survey data or establishment of new control points may begin.

Each new control point shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 2 hours. These time differentials are required in order to ensure significantly different satellite geometry. Each of these occupations shall consist of either one measurement of 180 epochs, or three sequential measurements of 60 epochs each.

Each topographic data point shall consist of a single measurement of at least 5 epochs.

Additional Notes:
All subsequent survey data shall be tied to the same legacy control point.

B. Placing the Base receiver on a control point within a local control network:

This method may be used when the current survey is to be tied to the control network of a previous survey, and may eliminate the need for Site Calibration.

After a link is established between Base and Rover, two additional points within the network are checked and stored as described above, and if these additional control points meet acceptable tolerance for the current survey and it is apparent that no rotation exists, collection of topographic survey data or establishment of new control points may begin.

Each new control point shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 2 hours. These time differentials are required in order to ensure significantly different satellite geometry. Each of these occupations shall consist of either one measurement of 180 epochs, or three sequential measurements of 60 epochs each.

Each topographic data point shall consist of a single measurement of at least 5 epochs.

Additional Notes: All survey data need not be tied to the same point within the existing project control network, however if the Base Station is moved, check shots must be measured and stored as above to two additional existing network control points, one of which being the first Base point.

Additional Field Procedures

- A known point shall be checked at the beginning and end of each session. Prudent practice would indicate additional checks, particularly after initialization is lost. Check shots should be coded with a unique numbering system which makes them easy to sort and verify. For example, a check shot on point #207 could be named "CHK207".
- Conditions which may generate multipath or obstruct view of the satellites, such as overhead power lines, nearby trees, or adjacent buildings, should be avoided.
- GPS receivers occupying the Base position shall be mounted on a tripod/tribrach configuration. Receiver HI is measured two times, one measurement in feet and one

in meters, and a unit conversion applied to verify the HI **before** the receiver is moved.

- Roving GNSS receivers occupying new or existing control points may be mounted either on a tripod/tribrach configuration or on a standard layout rod with supporting bipod. If a layout rod is used, the session should be split, with the rod being turned 180 degrees between sessions, so as to cancel out error introduced by improper adjustment of the plumb bubble.
- Topographic data points may be collected using a standard layout rod without the aid of a supporting bipod.
- Elevation mask shall be set to 15 degrees.
- Data should not be collected when PDOP exceeds 5.0.
- While the receiver is logging data, a full description of the physical monument is recorded and a digital image is captured.
- Occupation session data is recorded on an "RTK GPS Set-Up Sheet".
- Considerable care should be given to point naming conventions. Each time a point is occupied, it is to be given the **same** name. There is no need to use A, B, C etc. for subsequent occupations of the same point.

Important Note:
If the data link utilizes a UHF/VHF radio with an output of greater than 1 watt, a Federal Communications Commission (FCC) license is required. A copy of the license shall be stored in the receiver case and must be presented to FCC personnel upon request. All FCC rules and regulations shall be adhered to when performing an RTK survey. These include but are not limited to the following:

- Title 47, Code of Federal Regulations (CFR) part 90, Section 173 (Title 47 CFR 90.173): Obligates all licensees to cooperate in the shared use of channels
- Title 47 CFR 90.403: Requires licensees to take precautions to avoid interference, which includes monitoring prior to transmission
- Title 47 CFR 90.425: Requires that stations identify themselves prior to transmitting

Monumentation

Monuments set as control points during the course of a GNSS survey shall meet the following requirements:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of an iron pipe with a tag or disk, or a rebar with an aluminum cap. Rebar must be set a minimum of 3 inches

below the ground surface.

- All tags/washers/disks/caps referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge, and shall also be stamped "CP" or "CONTROL POINT".
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe or rebar.

Adjustment of the Network

All GNSS data shall be adjusted by least squares adjustment software, in conformance with Chapter 12 – Network Processing. Note that although an RTK network adjustment may be performed using only GNSS vectors (stand-alone), combining conventional traverse data with GNSS vectors will result in a network with higher relative positional accuracy.

Statistical analysis of the network adjustment shall be performed to ensure that a minimum combined (relative) positional accuracy of **1:10,000** has been achieved for all connected monument pairs (when RTK data is combined with conventional traverse data) or adjacent monument pairs (for stand-alone RTK data). Although this computation is automatically performed in most network adjustment software, the formula for this computation is shown in "Appendix A, Section 2."

Connections of very short distances often will not meet this **1:10,000** standard. An alternative standard for distances of less than **330** feet is shown in "Appendix A, Section 3."

In the event one or more pairs of monuments fail to pass these relative positional accuracy criteria, the network adjustment shall be reviewed and a determination made by the Senior Land Surveyor (or Project Manager) as to whether or not additional observations will made in order to improve geometry, increase redundancy, or further isolate errors.

Important Note:

Once a network has been adjusted and coordinates are reported to another entity (e.g.: Boundary Analysis Unit or Mapping Unit), these coordinates shall be deemed final. Should supplemental control or boundary ties be needed, the primary coordinates shall be fixed in subsequent adjustments. Only in the event that erroneous data is discovered will previously reported coordinate values be changed.

Appendix A – Formulas

1. Minimum spacing for new control points to be positioned using RTK GNSS can be computed using the following formula:

D = 10,000 x √ (2 x { [(1.96)(a)]2 + [(1.96)(b)]2 + c2 })
where:

- D = minimum spacing (in feet) between RTK occupation stations
- a = manufacturer's millimeter rating at a 68% confidence level, (converted to feet)
- b = manufacturer's ppm rating at a 68% confidence level, times the average distance (in feet) from legacy control stations, and divided by 1,000,000
- c = estimated receiver positioning error (rod plumb or tribrach errors), commonly estimated to be 0.007 feet
- 1.96 = the multiplier from a 68% confidence level (1 sigma) to a 95% confidence level (2 sigma)

2. All connected monument pairs shall pass the following mathematical test:

D ÷ √ (x2 + y2) ≥ 10,000 (or ≥ 20,000 where required above)
where:

- D = distance (in feet) between the pair of monuments being examined
- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

3. Connections of very short distances often will not meet the **1:10,000** standard defined by the formula in Section 2 above. An alternative standard for distances of less than 330 feet follows:

√ (x2 + y2) ≤ 0.033 feet
where:

- x = error ellipse semi-major axis for monument #1 (at 95% confidence)

- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

Appendix B - Site Calibration Procedure

A Site Calibration establishes a relationship between the observed WGS84 coordinates and the local grid coordinates.

General Requirements:

- The control stations shall be selected so as to create a polygon which fully encompasses the project area (see Figure 1). Selected control stations must be of the same epoch date as the current project and be located no more than six miles from the CRTN Station or Base Station.
- Conditions which may generate multipath or obstruct view of the satellites, such as overhead power lines, nearby trees, or adjacent buildings, should be avoided.
- Elevation mask shall be set to 15 degrees.
- Each occupation shall consist of either one measurement of 180 epochs, or three sequential measurements of 60 epochs each.
- Upon computation of the Site Calibration, a control station with residual values greater than those defined below shall be discarded and another control station shall be used in place of this outlier.
- All subsequent measurements and staking activities shall use the same CRTN Base Station or Base position as was used to generate the Site Calibration.

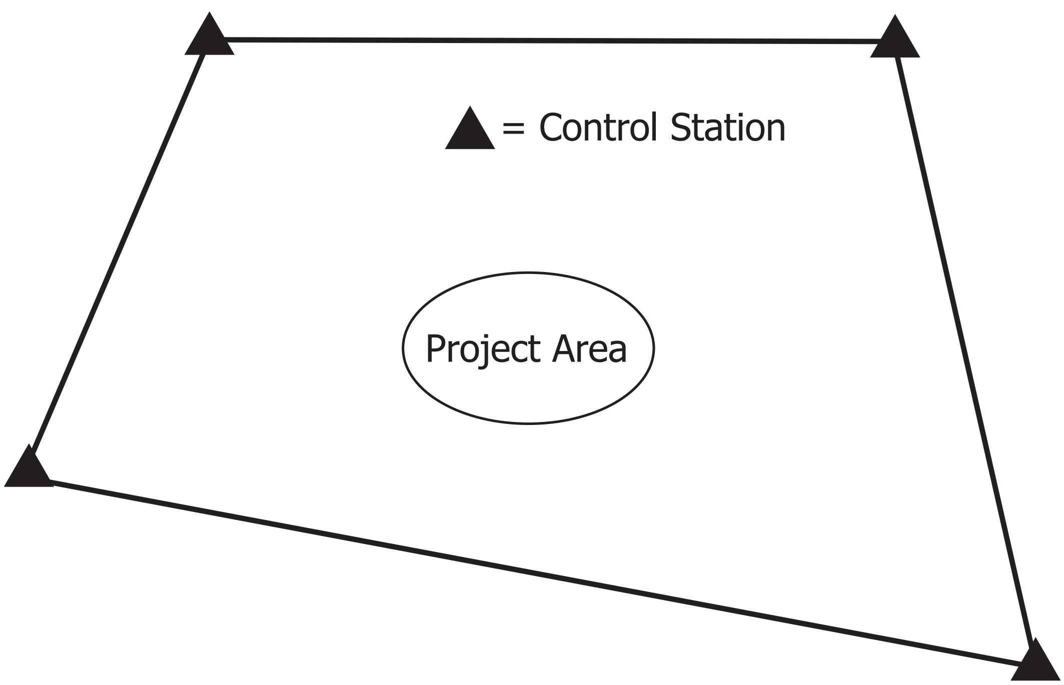


Figure 1 – Control Point Selection

2D Site Calibration:

- A minimum of 4 horizontal control stations shall be included in a 2D Site Calibration.
- Each horizontal control station shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 2 hours. These time differentials are required in order to ensure significantly different satellite geometry.
- The stations in a 2D Site Calibration shall not exceed a horizontal residual of 0.07 feet

3D Site Calibration:

- In addition to the requirements described above for a 2D Site Calibration, the following requirements shall be met for a 3D Site Calibration:
- A minimum of 5 vertical (or 3D) control stations shall be included in a 3D Site Calibration. To avoid creation of a distorted or tipped plane, the stations selected must have been tied together with one common leveling circuit. An alternative to this requirement is to collect data on these 5 vertical control stations but include just one of them in the Site Calibration. Analysis of the data will determine which vertical control station represents a best-fit solution for the project. This may be a better alternative when working with vertical control that has not been recently tied together (OC Survey Benchmarks).
- Each vertical control station shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 4 hours.
- The stations in a 3D Site Calibration shall not exceed a vertical residual of 0.10 feet.

CHAPTER 3
CONSTRUCTION STAKING

Policy Statement

Any survey which involves construction staking shall conform to the specifications as defined in this document.

General Note

These procedures are to be used in conjunction with the OC Survey Construction Staking Guide.

Preparation and Organization

Before construction activities commence, a thorough search of the site shall be performed in order to identify any existing monumentation (other than project specific construction control) which may be destroyed during construction. All provisions of Section 8771 of the Business and Professions Code (Land Surveyors' Act) shall be followed with regard to preservation of this existing monumentation.

Horizontal and vertical control used for construction staking should be based upon the control scheme used in gathering topographic data for design. In the event that this control scheme is not available, considerable effort shall be expended to ensure that new control set is of the same horizontal and vertical datum as the design data. Where identified on the construction plans, hard-surface conforms (joins) to existing infrastructure shall be verified well before staking operations begin.

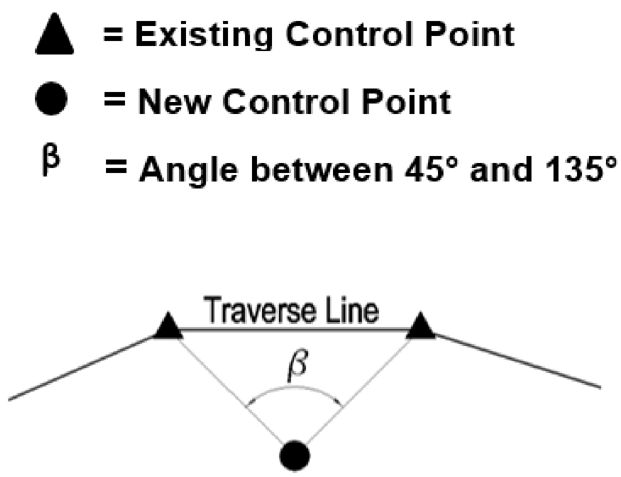
As construction activities commence and continue, original project control points often are destroyed and new points will need to be set. Some distinction needs to be made to identify the pedigree of the control so that degradation of integrity can be predicted. A point naming convention shall be established which clearly groups control points by their expected accuracy. Following is a list of typical control pedigrees, in order of decreasing integrity:

- First generation (original) control points
- Second generation control points, those set from first generation control
- Third generation control points, those set from second generation control, and so on
- Control points set by two-point resection**: These points should not be used to set additional control, and under no circumstances are points set by resection to be used to perform subsequent resections.

- Temporary points set for one specific, minor purpose, using weaker geometric principles: These points shall not to be used in any subsequent staking or control operations.

****Note:** Establishing control by two-point resection is permitted, provided a third control point is immediately checked to confirm the mathematical solution. The weakness of the two-point resection lies in the fact that error within the control used to establish the resection is greatly magnified and projected onto the resultant solution. The point to be established should be positioned so that the resultant horizontal angle between the fixed control points measures between 45 and 135 degrees (see Figure 1). Keep in mind that as control integrity naturally degrades from generation to generation, the two-point resection becomes a less desirable option.

Figure 1 – Two-Point Resection



Upon completion of the project, memorialization of project control and the setting of final monumentation and resetting of any destroyed monumentation should be based upon the earliest generation control which remains.

Vertical Control Procedures

Original control shall be elevated using differential leveling procedures and be in conformance with [Chapter 4 – Differential Leveling](). Elevations of subsequent control may be derived by trigonometric principles, provided the points are traversed through, double determined, or set by two-point resection, with acceptable mathematical vertical closures observed.

Staking Procedures – Conventional Instruments (total stations)

Whether or not stakeout activities are performed robotically, a tripod/tribrach target should be used for backsight orientation. After the instrument is oriented and the measurement to the backsight is recorded, a third control point is staked out, read, and recorded using the layout rod that will be used for setting of stakes. This provides assurance that the prism offset and rod HI measurement are correct. Check shots should be coded with a unique numbering system which makes them easy to sort and verify. For example, a check shot on point #207 could be named "CHK207".

Before proceeding with setting of stakes, check into stakes set previously, if any exist within a reasonable radius. Where applicable, conforms (joins) to existing infrastructure shall be checked.

In order to prevent degradation of the horizontal and vertical accuracy of stakes, stakeout distances should be limited as follows:

- Limit distances to a maximum of 400 feet when setting stakes for curb, walls, and other features generally requiring horizontal and vertical tolerances of 0.03 feet and 0.02 feet respectively.
- Limit distances to a maximum of 800 feet when setting stakes for rough grade and other features generally requiring horizontal and vertical tolerances of 0.1 feet or greater.

After completion of staking, a check shot to a control point or a previously set stake shall be read and recorded.

Note regarding staking of features with very low rates of flow: Gravity flow pipelines, concrete curb, and the lowest flow line feature of concrete channels, which have a design flow rate of **≤1.0%**, shall be elevated using differential leveling procedures.

Staking Procedures – RTK GNSS

Use of RTK GNSS when performing construction staking is permissible only under the guidelines defined below:

- <u>Clearing Limits, Grading Limits, Construction Easements, ESA Fencing:</u> Provided the project coordinate system and epoch date are the same as that of the base station, stakes of this nature do not require Site Calibration. If a Site Calibration is not performed, known point check shots must be within 0.15 feet horizontally. No elevations will be reported.

- Slope Stakes, Rough Grade, Top and Toe Stakes, Pressurized Water Lines, Dry Utility Trenches: Stakes of this nature require a 3D Site Calibration (refer to "Appendix A" for Site Calibration procedure). Elevations and cuts are recorded to the nearest 0.1 feet. Note that stakes set for fire hydrants which reference top of curb elevations may be staked with RTK but must be elevated with a conventional instrument (a total station or level).
- Storm Drain Pipelines: Only the horizontal component of storm drain pipelines may be staked with RTK. Stakes must then be elevated with a conventional instrument. A 2D Site Calibration is required.
- Sewer Pipelines: Only the horizontal component of sewer lines may be staked with RTK. Stakes must then be elevated with a conventional instrument. A 2D Site Calibration is required.
- Concrete Channels: Stakes for concrete channels shall NOT be set with RTK.
- Subgrade: Only the horizontal component of subgrade may be staked with RTK. Stakes must then be elevated with a conventional instrument. A 2D Site Calibration is required.
- Major Structures, Retaining Walls, Curb and Gutter, Catch Basins: Stakes for major structures (bridge abutments, piers, or columns), retaining walls, curbs, and catch basins shall NOT be set with RTK.

In all cases, a known point shall be checked before, at multiple times during, and again after staking. Check shots should be coded with a unique numbering system which makes them easy to sort and verify. For example, a check shot on point #207 could be named "CHK207".

Quality Control/Assurance

Before leaving the job site, a stakeout tolerance report shall be generated from the data collector job-file to ensure that all stakes have been set within tolerances defined by the OC Survey Construction Staking Guide.

Archiving of Survey Data

Electronic cut sheets (examples can be found here) shall be transferred to the Party Chief's workstation along with associated stakeout tolerance reports (examples can be found here). Sketches or hand-recorded notes shall be scanned and filed electronically. Data collector Job files shall also be transferred to the Party Chief's workstation and either a master point plot or multiple sub point plots maintained for the duration of the project.

Appendix A - Site Calibration Procedure

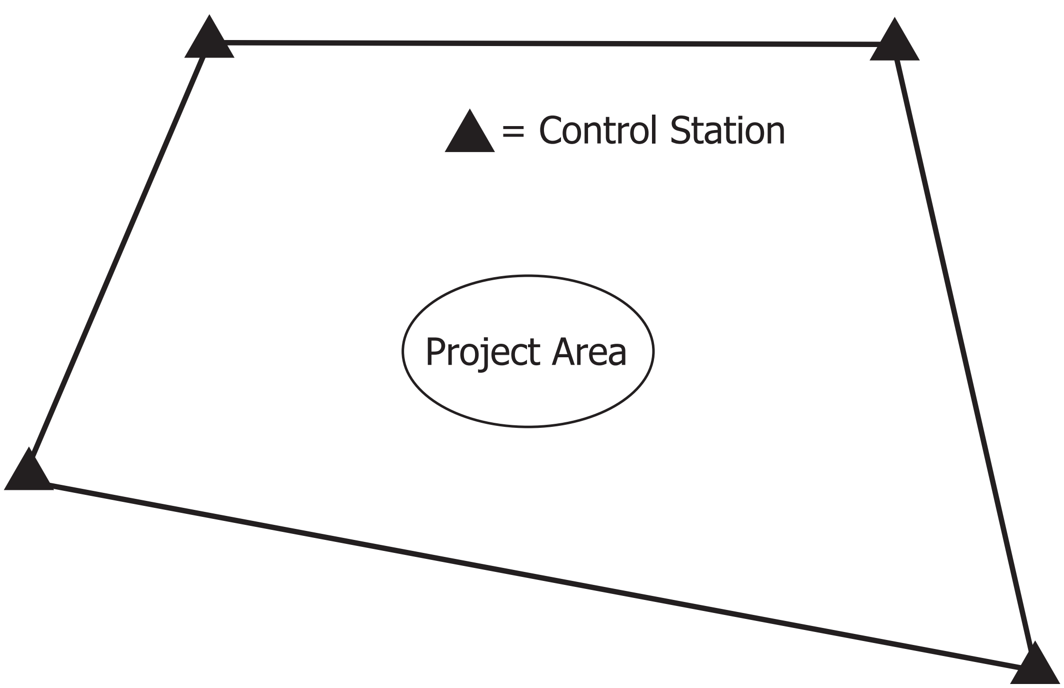
A Site Calibration establishes a relationship between the observed WGS84 coordinates and the local grid coordinates.

The procedures detailed below are specific to construction staking projects. See Chapter 2 – RTK GNSS for more general uses and procedures for Site Calibrations.

General Requirements:

- The control stations shall be selected so as to create a polygon which fully encompasses the project area (see **Figure 2**). Selected control stations shall be located no more than six miles from the RTN station or base station.
- Conditions which may generate multipath or obstruct view of the satellites, such as overhead power lines, nearby trees, or adjacent buildings, should be avoided.
- Elevation mask shall be set to 15 degrees.
- Each occupation shall consist of either one measurement of 180 epochs, or three sequential measurements of 60 epochs each.
- Upon computation of the Site Calibration, a control station with residual values greater than those defined below shall be discarded and another control station shall be used in place of this outlier.
- All subsequent measurements and staking activities shall use the same RTN base station or base position as was used to generate the Site Calibration.

Figure 2 – Control Station Selection



2D Site Calibration:

- A minimum of 4 horizontal control stations shall be included in a 2D Site Calibration.
- Each horizontal control station shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 2 hours. These time differentials are required in order to ensure significantly different satellite geometry.
- The stations in a 2D Site Calibration shall not exceed a horizontal residual of 0.07 feet

3D Site Calibration:

In addition to the requirements described above for a 2D Site Calibration, the following requirements shall be met for a 3D Site Calibration:

- A minimum of 5 vertical (or 3D) control stations shall be included in a 3D Site Calibration. To avoid creation of a distorted or tipped plane, the stations selected must have been tied together with one common leveling circuit. An alternative to this requirement is to collect data on these 5 vertical control stations but include just one of them in the Site Calibration. Analysis of the data will determine which vertical control station represents a best-fit solution for the project. This may be a better alternative when working with vertical control that has not been recently tied together (OC Survey Benchmarks).
- Each vertical control station shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 4 hours.
- The stations in a 3D Site Calibration shall not exceed a vertical residual of 0.10 feet.

CHAPTER 4 DIFFERENTIAL LEVELING

Policy Statement

Any survey performed for the purpose of establishing elevations by differential leveling shall conform to the specifications as defined in this document.

Third Order Leveling

The following requirements are specific to leveling projects which establish elevations of control points used for Construction Staking, Topographic Surveys, LiDAR Surveys, and Scour Study Surveys, and follow Third Order leveling procedures as defined by the Federal Geographic Data Committee (FGDC):

Equipment Selection:

- Either a digital bar-code level or pendulum type optical level may be used
- Tripods may be of any style
- Leveling rods may be multi-piece bar code rods or graduated Philadelphia rods
- Turning points may consist of turning plates ("turtle" or "puck"), wooden stakes, or nails

Technical Requirements:

- Collimation of the level (Peg Test) shall be performed at the start of each project (see "Appendix A" below)
- Maximum collimation error: 0.10 mm/meter (20 arc-seconds)
- Maximum benchmark spacing: 3 kilometers
- Maximum sighting distance: 90 meters
- Minimum ground clearance: 0.5 meters
- Maximum imbalance – per turn: 10 meters
- Maximum imbalance – per section: 10 meters
- Maximum misclosure per section (in meters): 0.012 x √ length in kilometers
- Maximum misclosure per loop (in meters): 0.012 x √ length in kilometers

Project Benchmark Requirements:

All projects shall include a level circuit which is tied to a minimum of two benchmarks from the OCS Vertical Control Network. It is acceptable for the level circuit to include benchmarks from different level lines, e.g. benchmark(s) from the **3C** line may be paired with benchmark(s) from the **SA** line. However it is advisable to only include benchmarks which have been leveled in the same year.

Note: Regardless of the number of benchmarks included in the level circuit and the misclosure between benchmarks, the level circuit must form a closed loop, closing back on the first point of the circuit.

Elevations of Supplemental Control Points:

Elevations of subsequent supplemental control points may be derived by trigonometric principles, provided the points are traversed through, double determined, or set by two-point resection, with acceptable mathematical vertical closures observed.

Geodetic Leveling

The following requirements are specific to geodetic leveling projects which establish elevations of benchmarks to be included in the OC Survey Geodetic Control Network, and follow Second Order Class I leveling procedures as defined by the Federal Geographic Data Committee (FGDC):

Equipment Selection:

- A digital bar-code level shall be used
- Tripods shall be of fixed-leg variety
- Leveling rods shall be one-piece invar rods (single or double scale) and use a 10 minute plumb bubble
- Stabilizing stays shall be used to support the leveling rods
- Turning points shall consist of turning plates ("turtle" or "puck") weighing a minimum of 15 pounds

Physical Procedural Requirements:

- Sections shall begin on two Second Order Class I (or better) benchmarks
- Sections shall end on two Second Order Class I (or better) benchmarks
- All sections shall be double-run, either with two independent runs or by the Single Run Double Simultaneous procedure (SRDS), also known as BFFB in Leica terminology
- Two leveling rods (labeled "A" and "B") shall be used in leap-frog fashion, with the same rod ("A" rod) taking off of and closing in on all benchmarks

Technical Requirements:

Collimation of the level (Peg Test) – shall be performed at the start of each day (see "Appendix A" below)

- Maximum collimation error: 0.05 mm/meter (10 arc-seconds)
- Average benchmark spacing: 1.6 kilometers
- Maximum benchmark spacing: 3 kilometers
- Minimum ground clearance: 0.5 meters
- Maximum sighting distance: 60 meters
- Number of readings per set: 3
- Maximum standard error of the mean per set: 0.0001 meters
- Maximum imbalance – per turn: 5 meters
- Maximum imbalance – per section: 10 meters
- Maximum misclosure per section (in meters): 0.006 x √ length in kilometers
- Maximum misclosure per loop (in meters): 0.006 x √ length in kilometers

Additional Deliverables for New Benchmarks:

- Approximate Latitude/Longitude
- Digital photograph of the monument
- Digital photograph of the location
- Written description of the monument
- Detailed description of the location with to-reach instructions

Character of New Benchmarks:

New benchmarks shall consist of a 4” diameter domed aluminum disk stamped “COUNTY OF ORANGE SURVEYOR BENCH MARK” (see Figure 1).

Figure 1 – Benchmark Disk

Location of New Benchmarks:
Disks shall be epoxied or cemented to fixed works in such a manner as to ensure stability. Following are possible benchmark locations, listed in order of descending stability:

- Bridge abutment, headwall, or other major structure
- Catch basin or underground vault
- Pre-cast concrete post

Adjustment of the Network

Second and Third Order differential leveling surveys, when run as a single loop or section, are adjusted by a straight-line interpolation process. Corrections for the closing error will be prorated to each benchmark between the controlling benchmarks.
When multiple leveling sections or loops interconnect to form a network, points common to two or more loops shall be adjusted by least squares adjustment software, in conformance with Chapter 12 – Network Processing.

Important Note:
Once a network has been adjusted and elevations are reported to another entity (e.g.: Mapping Unit), these elevations shall be deemed final. Should supplemental control be needed, the primary elevations shall be fixed in subsequent adjustments. Only in the event that erroneous data is discovered will previously reported elevation values be changed.

Appendix A – Peg Test Procedures

Collimation of the level is accomplished by performing a Peg Test. There are two main methods for performing a Peg Test, both of which are supported by the internal software on Leica leveling instruments:

1. Method 1 – "A 1 2 B"

- Two rod positions ("A" and "B") are established 60 meters apart, +/- 1 meter
- Two instrument positions are established ("1" and "2") at 1/3 and 2/3 of the total distance, +/- 1 meter
- Instrument occupies position "1" and measures first to the rod at "A" (measurement "A1") and then to the rod at "B" (measurement "B1")
- Instrument occupies position "2" and measures first to the rod at "B" (measurement "B2") and then to the rod at "A" (measurement "A2")
- Collimation is computed and displayed in arc-seconds (1" of arc = 0.005 mm/meter)
- Compare computed collimation with maximum value defined for each Order in **"Technical Requirements"** above, and store the value if within tolerance

2. Method 2 – "A 1 B 2"

- Two rod positions ("A" and "B") are established 40 meters apart, +/- 1 meter
- Two instrument positions are established: position "1" at 1/2 of the total distance, +/- 1 meter, and position "2 " at 3 meters beyond (outside) rod position "B"
- Instrument occupies position "1" and measures first to the rod at "A" (measurement "A1") and then to the rod at "B" (measurement "B1")
- Instrument occupies position "2" and measures first to the rod at "B" (measurement "B2") and then to the rod at "A" (measurement "A2")
- Collimation is computed and displayed in arc-seconds (1" of arc = 0.005 mm/meter)
- Compare computed collimation with maximum value defined for each Order in **"Technical Requirements"** above, and store the value if within tolerance

CHAPTER 5
BOUNDARY SURVEYS

Policy Statement

Any survey which locates or establishes land boundaries, rights of way, or centerline alignments, collectively referred to herein as "boundary surveys", shall conform to the specifications as defined in this document.

General

Boundary surveys shall conform to a minimum combined (relative) positional accuracy of **1:10,000** (at a 95% confidence level, or 2 sigma), or a combined distance error of **≤ 0.033** feet for connection distances shorter than **330 feet**. Relative positional accuracy is a measure of the accuracy of point positions in relation to each other, and is not to be confused with the measure of traverse closure expressed as a ratio.

This **1:10,000** standard shall be met whether the survey is conducted by GNSS (static or RTK), conventional traverse (total station), or any combination thereof.

Following are guidelines for GNSS and conventional traverse methodology:

Static GNSS

Primary control for a boundary project may be established by static (or fast-static) GNSS procedures. While a network adjustment may be performed using only GNSS vectors (stand-alone), combining conventional traverse data with GNSS vectors will sometimes result in a network with higher relative positional accuracy.

Design of the network and occupation scheme will be determined by the Party Chief in conformance with Chapter 1 – Static GNSS. When selecting points to be included in the static network, consideration must be given as to strength of figure and adequate spacing. The minimum allowable spacing for points in stand-alone networks shall be dictated by the following criteria:

- Trimble R10 receivers, rated for static surveys at 3mm + 0.5 ppm at 68% confidence level (1 sigma): a minimum spacing of **500 feet** when tied to CGPS stations at an average distance of 32,000 feet, and a minimum spacing of **300 feet** when tied to primary project control or legacy control at an average distance of 4,000 feet
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

RTK GNSS

RTK is generally not to be used as a stand-alone measurement tool when performing a boundary survey. RTK is best used to bolster the control network, not define it. In order to ensure realization of the **1:10,000** criteria, the network shall be adjusted using RTK measurements together with conventional traverse data.

RTK occupation points are selected in such a way as to maximize strength of figure, while leaving the bulk of the data to be captured by conventional traverse. The occupation scheme will be determined by the Party Chief in conformance with Chapter 2 – RTK GNSS.

The minimum recommended spacing for points in RTK surveys shall be dictated by the following criteria:

- Trimble R10 receivers, rated for RTK surveys at 8mm + 1 ppm at 68% confidence level (1 sigma): a minimum spacing of **1200 feet** when tied to CRTN stations at an average distance of 32,000 feet; a minimum spacing of **700 feet** when tied to local project control in a base-rover configuration at an average distance of 4,000 feet
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

Conventional Traverse (Total Station)

The method selected for traversing and tying in boundary monuments using a total station shall be limited to the following options, listed in order of descending accuracy:

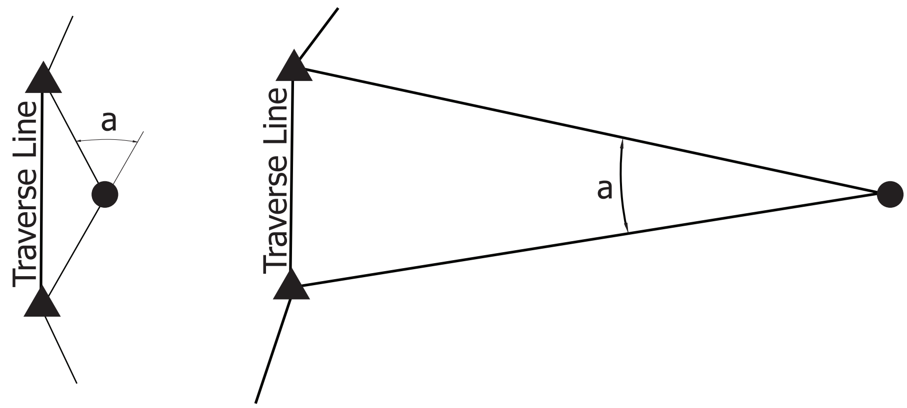
- **Traverse through each monument:** This option generally delivers the highest accuracy. This method may be impractical or impossible, as many monuments lie adjacent to obstructions or in areas of heavy vehicular traffic. In some cases, poor geometry may be introduced into an otherwise sound network.
- **Double determination from two or more control stations:** This is an acceptable method when monuments lie adjacent to obstructions, or in the event of short sighting distances or multiple flat horizontal angles. If a monument is tied to only two control stations, the resulting deflection angle or included angle should be greater than 15 degrees (see Figure 1).

Figure 1 – Double Determination

▲ = Control Station

● = Boundary Monument to be Positioned

a = Angle Greater than 15°

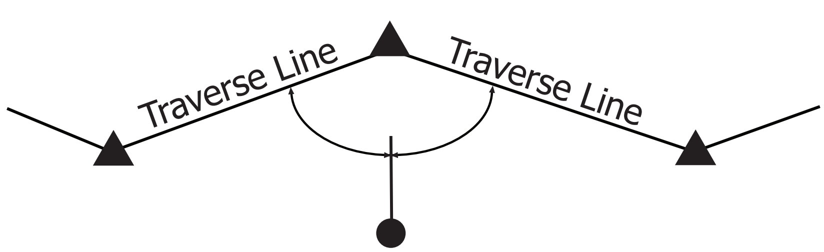


- **Double backsight:** This is achieved by observing a monument from a single control station by making two or more independent observations, each using a different backsight point (see **Figure 2**). Double backsight may be the preferable method in the instance of a monument falling within dense foliage or under a wall footing, where there is a narrow window through which to observe the monument. Although this method does eliminate the possibility of a blunder and provides better modeling of the network adjustment than a simple single determination, it is to be used as a last resort. Other methods shall be deemed impossible or degrading to the network before double backsight is employed. If this method is to be used, the instrument setup must broken-down and re-erected between measurements.

Figure 2 – Double Backsight

▲ = Control Station

● = Boundary Monument to be Positioned



General Procedural Notes:

It is preferable to include as much redundancy as possible; if points are intervisible, measure between them. When possible, each pair of adjacent boundary monuments that are intervisible should be directly measured, as this best represents the boundary line being retraced. This is especially critical in the case of monuments that are close together, such as the radius point and centerline terminus of a cul-de-sac, or a jog in a property line or centerline. Very short lines of this nature should be chained, and the resultant distance weighted appropriately in the network adjustment.

Field measurements shall meet the following specifications:

- Horizontal Angles: Minimum of two direct (face 1) and two reverse (face 2) with a maximum residual of 5 seconds; exception granted for sights closer than 300 feet.
- Distances: Measured to backsight and foresight; minimum of two direct and two reverse with a maximum residual of 0.007 feet.

Monumentation

Monuments set during the course of a boundary survey shall meet the following criteria:

Boundary Corners

Monuments set at boundary corners for Tract Maps or Parcel Maps, or on any interior lot or parcel lines to be further subdivided, or for future subdivision Records of Survey:

- Monuments which fall in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall in the surface of asphalt paving shall consist of a durable spike (minimum 4 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of a 2 inch diameter iron pipe with a tag or disk.
- All tags/washers/disks referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe.

Lot and Parcel Corners

Monuments set at lot and parcel corners for Tract Maps, Parcel Maps, Records of Survey, Corner Records, Lot Line Adjustments, and Certificates of Compliance:

- Monuments which fall in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall in the surface of asphalt paving shall consist of a durable spike (minimum 4 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of a 1 inch diameter iron pipe with a tag.
- All tags/washers referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe.

Street Centerline Points

Monuments set at street intersections, the controlling points along the centerlines of streets, and where boundary lines are produced to intersect street centerlines:

- Monuments which fall in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall in the surface of asphalt paving shall consist of a durable spike (minimum 4 inches in length) with a washer. A Survey Monument Type "A" (monument well), per *OC Public Works Standard Plan 1405*, may be set in lieu of spike and washer described above. The number and location of Type "A" monuments shall be as directed by the County Surveyor.
- Monuments which fall in non-paved areas shall consist of a 1 inch diameter iron pipe with a tag. A Survey Monument Type "B", per OC Public Works Standard Plan 1406, may be set in lieu of iron pipe and tag described above. The number and location of Type "B" monuments shall be as directed by the County Surveyor.
- All tags/washers referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe.

Reference Points (Tie Points)

Monuments which represent tie points set for the purpose of monument perpetuation and/or preservation:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug and countersunk so as to be flush with the concrete surface.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of a 1 inch diameter iron pipe with a tag.
- All tags/washers referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe.

Control Points

Monuments set as control points during the course of a survey:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of an iron pipe with a tag or disk, or a rebar with an aluminum cap. Rebar must be set a minimum of 3 inches below the ground surface.
- All tags/washers/disks/caps referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge, and shall also be stamped "CP" or "CONTROL POINT".
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe or rebar.

Adjustment of the Network

All GNSS and conventional data shall be adjusted by least squares adjustment software, in conformance with Chapter 12 – Network Processing.

Statistical analysis of the adjustment shall be performed to ensure that a minimum combined (relative) positional accuracy of **1:10,000** has been achieved for all connected monument pairs, and for all adjacent monument pairs (consecutive points along an alignment or boundary line), whether or not they are connected within the network. Although this computation is automatically performed in most network adjustment software, the formula for this computation is shown in "Appendix A, Section 2."

Connections of very short distances often will not meet this **1:10,000** standard. An alternative standard for distances of less than **330** feet is shown in "Appendix A, Section 3."

In the event one or more pairs of monuments fail to pass these relative positional accuracy criteria, the network adjustment shall be reviewed and a determination made by the Senior Land Surveyor (or Project Manager) as to whether or not additional observations will made in order to improve geometry, increase redundancy, or further isolate errors.

Important Note:

Once a network has been adjusted and coordinates are reported to another entity (e.g.: Boundary Analysis Unit or Mapping Unit), these coordinates shall be deemed final. Should supplemental control or boundary ties be needed, the primary coordinates shall be fixed in subsequent adjustments. Only in the event that erroneous data is discovered will previously reported coordinate values be changed.

Equipment Selection and Use

Specific notes on equipment selection and usage are outlined below:

- Static GNSS Surveys: Receivers shall be mounted on a tripod/tribrach configuration. One acceptable alternative mounting is a "four-legged" fixed height or locking-pin rod. This rod shall have three support legs and a center leg which freely turns 360 degrees. A standard layout rod with supporting bipod shall NOT be used for static GNSS occupations.

- RTK Surveys: Receivers may be mounted on a standard layout rod with supporting bipod. If this configuration is used, the session should be split, with the rod being turned 180 degrees between sessions, so as to cancel out error introduced by improper adjustment of the plumb bubble.
- Conventional Surveys: Measurements should be made to a prism mounted on a tripod/ tribrach configuration, unless impractical or impossible. If a layout rod is used in place of a tripod mounted target, a bipod or other suitable stabilizing element should be employed. The rod should be turned 180 degrees between measurements, so as to cancel out error introduced by improper adjustment of the plumb bubble. A hand held "peanut prism" may be placed directly on a monument, provided a plumb bubble is incorporated into the prism assembly. This prism should be turned 180 degrees between measurements as well. Prisms designated as 360 degree prisms are NOT to be used for a boundary survey. Atmospheric PPM correction shall be computed and applied as needed throughout the survey, however when measured distances exceed 400 feet, temperatures shall be measured with a thermometer, and not simply estimated.

Appendix A – Formulas

1. Minimum spacing for new control points to be positioned using GNSS can be computed using the following formula:

$$\mathbf{D = 10{,}000 \times \sqrt{\;}(2 \times \{ [(1.96)(a)]2 + [(1.96)(b)]2 + c2 \})}$$

where:

- D = minimum spacing (in feet) between static or RTK occupation stations
- a = manufacturer's millimeter rating at a 68% confidence level, (converted to feet)
- b = manufacturer's ppm rating at a 68% confidence level, times the average distance (in feet) from legacy control stations, and divided by 1,000,000
- c = estimated receiver positioning error (rod plumb or tribrach errors), commonly estimated to be 0.007 feet
- 1.96 = the multiplier from a 68% confidence level (1 sigma) to a 95% confidence level (2 sigma)

2. All connected monument pairs shall pass the following mathematical test:

D ÷ √ (x2 + y2) ≥ 10,000 (or ≥ 20,000 where required above)

where:

- D = distance (in feet) between the pair of monuments being examined
- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

3. Connections of very short distances often will not meet the **1:10,000** standard defined by the formula in Section 2 above. An alternative standard for distances of less than 330 feet follows:

√ (x2 + y2) ≤ 0.033 feet

where:

- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

CHAPTER 6
TOPOGRAPHIC SURVEYS

Policy Statement

Any survey performed for the purpose of collecting topographic data shall conform to the specifications as defined in this document.

Preparation

Before planning the survey, a meeting between the Party Chief and/or Senior Land Surveyor and the Requestor shall be conducted on the job site. During this meeting, the Requestor will explain specific project requirements as detailed on the Initial Survey Request. It is the responsibility of the Party Chief to listen attentively and take notes, and to bring up any questions or concerns. If any ***significant*** changes to the scope of the project are identified and agreed upon during this meeting, the Requestor must complete an Additional Survey Request.

Accuracy Requirements

Topographic data points should be located within the following tolerances, relative to project control ("local accuracy"):

- Data points representing hardscape features (concrete, asphalt, underground vaults) should be located within +/- 0.03 feet horizontally and +/- 0.02 feet vertically
- Data points representing utility appurtenances (pull boxes, manholes, fire hydrants) should be located within +/- 0.15 feet horizontally and +/- 0.02 feet vertically; note that utility appurtenances may be collected with RTK GNSS, provided they are omitted from the DTM (see section below entitled "RTK GNSS" for more details)
- Data points representing original ground features (dirt breaklines, spot elevations) should be located within +/- 0.15 feet horizontally and +/- 0.1 feet vertically
- Data points representing landscaping features (trees, shrubs) should be located within +/- 0.5 feet horizontally and +/- 0.1 feet vertically

Field Methodology

This document establishes acceptable methodology for topographic surveys incorporating conventional instruments (total stations) and RTK GNSS. See Chapter 7 - LiDAR Surveys and Chapter 10 – Small Unmanned Aerial Vehicle (sUAV) Surveys for alternate methods of data collection.

See sections below related to establishing the control network for details on the horizontal and vertical control network upon which the topographic survey is based.

General Methodology:

The following general notes relate to data collected from any source:

- Instrument set-up information, point ranges used, edits needed, etc. shall be logged on a "Data Collection Set-Up Sheet".
- Data points shall include proper topo codes and attribute data from OC Survey's "Topo Code List."
- Spacing of data points shall be such as is necessary to capture an accurate representation of the horizontal and vertical geometry of all linear features and establish an accurate Digital Terrain Model (DTM) inclusive of all features and surfaces. Determination of point spacing is made by the Party Chief on a project by project basis, however spacing shall not exceed 25 feet for points representing hardscape features, or 50 feet for points representing original ground features.
- Care must be taken to ensure that individual data points do not misrepresent or corrupt the DTM - for example a tree may lift the ground elevation in the area of the trunk. Shots of this nature should be omitted from the DTM.
- When capturing data points within any soft surface, e.g. sand or mud, the layout rod shall be equipped with a "shoe" or other blunt object designed to keep the rod from sinking beneath the surface.

Conventional Instruments:

The following notes are specific to data collected with a conventional instrument:

- After the instrument is oriented and the measurement to the backsight is recorded, a third control point is staked out, read, and recorded using the layout rod that will be used for collection of data points. This provides assurance that the prism offset and rod HI measurement are correct. Check shots should be coded with a unique numbering system which makes them easy to sort and verify. For example, a check shot on point #207 could be named "CHK207".
- In order to prevent degradation of the horizontal and vertical accuracy of data points, measurement distances should be limited to a maximum of 400 feet when collecting hardscape features and 800 feet when collecting original ground or landscaping features.
- When collecting hardscape features, the prism rod HI should be limited to 10 feet.

- Some features, such as traffic signal poles and trees, cannot be precisely occupied by the prism rod. It is at the discretion of the Party Chief as to when these features will be collected using an “offset routine”, which computes an approximate position of the center of the feature. Typically, signal poles and other features having a direct impact on curb-ramps and sidewalk widths are collected in this manner. Trees and similar features can often be collected by simply placing the prism rod next to the feature.

RTK GNSS:
The following notes are specific to data collected with RTK GNSS (RTK):

- Refer to Chapter 2 - RTK GNSS for detailed policy on the use of RTK.
- A known point shall be checked at the beginning and end of each session. Prudent practice would indicate additional checks, particularly after initialization is lost. Check shots should be coded with a unique numbering system which makes them easy to sort and verify. For example, a check shot on point #207 could be named “CHK207”.
- Only data points representing original ground features (dirt breaklines, spot elevations), utility appurtenances, and landscape features (trees, shrubs) may be collected with RTK (note that utility appurtenances collected with RTK shall be omitted from the DTM). Hardscape features shall be collected with a conventional instrument.

Monumentation

Monuments set as control points during the course of a topographic survey shall meet the following criteria:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or “MAG” nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of an iron pipe with a tag or disk, or a rebar with an aluminum cap. Rebar must be set a minimum of 3 inches below the ground surface.
- All tags/washers/disks/caps referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge, and shall also be stamped “CP” or “CONTROL POINT”.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe or rebar.

Office Workflow

The following office workflow is to take place as the fieldwork progresses:

- Field data should be downloaded at the end of each day. Data shall be reviewed and edited by or under the supervision of the Party Chief. Care must be taken to resolve crossing breaklines, errors in the surface, or miscoded points. Edits shall be recorded on a "Topo Dat File Edit Log" form.
- This edited data shall be delivered to the Mapping Unit (placed on the Field Survey Server) in blocks of no larger than three days of fieldwork.
- Note that the files themselves are not delivered by email. Files shall be copied to the Field Survey Server and organized as described in Chapter 13 – Preparation of the Field Note Package. A link to the file location is emailed to the Mapping Unit.
- Data is delivered in Starnet **DAT** format (see "Adjustment of the Network" below for more details on generating the **DAT file**). The Starnet project file (**.SNPROJ**) containing all project settings is also delivered, and as new field data is collected, the Mapping Unit and the Party Chief will each maintain a running amended adjustment as an additional layer of QA/QC.
- Along with the Starnet data, the Party Chief shall deliver data collector job files (.JOB), copies of the Data Collection Set-Up Sheets, and field notes or sketches.
- The Mapping Unit will process the data, run surface modeling, and add the data to the master project.
- After completion of the fieldwork and topo processing, the Mapping Unit will provide the field crew with a plot of the entire project. The Party Chief shall walk the jobsite with the plot in hand, searching for errors or omissions, making corrections as needed.
- After final edits have been made, a plot is provided for the Senior Land Surveyor to review.

Establishing the Horizontal Control Network

Horizontal control for topographic surveys shall conform to a minimum combined (relative) positional accuracy of **1:10,000** (at a 95% confidence level, or 2 sigma), or a combined distance error of **≤0.033** feet for connection distances shorter than **330 feet**. Relative positional accuracy is a measure of the accuracy of point positions in relation to each other, and is not to be confused with the measure of traverse closure expressed as a ratio. This **1:10,000** standard shall be met whether the survey is conducted by GNSS (static or RTK), conventional traverse (total station), or any combination thereof.

The following are guidelines for GNSS and conventional traverse methodology:

Static GNSS:
Control for a topographic survey project may be established by static (or fast-static) GNSS procedures. While a network adjustment may be performed using only GNSS vectors (stand-alone), combining conventional traverse data with GNSS vectors will sometimes result in a network with higher relative positional accuracy.

Design of the network and occupation scheme will be determined by the Party Chief in conformance with Chapter 1 – Static GNSS When selecting points to be included in the static network, consideration must be given as to strength of figure and adequate spacing. The minimum allowable spacing for points in stand-alone networks shall be dictated by the following criteria:

- Trimble R10 receivers, rated for static surveys at 3mm + 0.5 ppm at 68% confidence level (1 sigma): a minimum spacing of 500 feet when tied to CGPS stations at an average distance of 32,000 feet, and a minimum spacing of 300 feet when tied to primary project control or legacy control at an average distance of 4,000 feet
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

RTK GNSS:
RTK is generally not to be used as a stand-alone measurement tool when performing a control survey. RTK is best used to bolster the control network, not define it. In order to ensure realization of the **1:10,000** criteria, the network shall be adjusted using RTK measurements together with conventional traverse data.

RTK occupation points are selected in such a way as to maximize strength of figure, while leaving the bulk of the data to be captured by conventional traverse. The occupation scheme will be determined by the Party Chief in conformance with Chapter 2 – RTK GNSS. The minimum recommended spacing for points in RTK surveys shall be dictated by the following criteria:

- Trimble R10 receivers, rated for RTK surveys at 8mm + 1 ppm at 68% confidence level (1 sigma): a minimum spacing of **1200 feet** when tied to CRTN stations at an average distance of 32,000 feet; a minimum spacing of **700 feet** when tied to local project control in a base-rover configuration at an average distance of 4,000 feet
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

Conventional Traverse (Total Station):

Conventional traversing may be used either as a stand-alone method or in combination with GNSS vectors when establishing control networks for topographic surveys.

Field measurements shall meet the following specifications:

- Horizontal Angles: Minimum of two direct (face 1) and two reverse (face 2) with a maximum residual of 5 seconds; exception granted for sights closer than 300 feet.
- Distances: Measured to backsight and foresight; minimum of two direct and two reverse with a maximum residual of 0.007 feet.

Establishing Vertical Control

For topographic surveys representing hardscape features (concrete, asphalt), elevations of points within the primary control network shall be established using differential leveling procedures. Leveling shall be referenced to a minimum of two vertical control points (benchmarks) and be in conformance with Chapter 4 – Differential Leveling.

Elevations of subsequent supplemental control may be derived by trigonometric principles, provided the points are traversed through, double determined, or set by two-point resection, with acceptable mathematical vertical closures observed.

Adjustment of the Network

Control Network:

All GNSS and conventional data shall be adjusted by least squares adjustment software in conformance with Chapter 12 – Network Processing.

Statistical analysis of the adjustment shall be performed to ensure that a minimum combined (relative) positional accuracy of **1:10,000** has been achieved for all connected monument pairs. Although this computation is automatically performed in most network adjustment software, the formula for this computation is shown in "Appendix A, Section 2".

Connections of very short distances often will not meet this **1:10,000** standard. An alternative standard for distances of less than **330 feet** is shown in "Appendix A, Section 3".

In the event one or more pairs of monuments fail to pass these relative positional accuracy criteria, the network adjustment shall be reviewed and a determination made by the Senior Land Surveyor (or Project Manager) as to whether or not additional observations will made in order to improve geometry, increase redundancy, or further isolate errors.

Topo Sideshots:

After the control network has been satisfactorily adjusted, topographic data points are added. Unique **DAT files** for each block of data (data representing 1 to 3 days of fieldwork) are created from the data collector files. Be sure to export data points in **sideshot** format. All necessary edits are made within the **DAT files**.

All topo sideshot **DAT files** - original and edited - are delivered to the Mapping Unit, along with **DAT file/s** representing the project horizontal and vertical control network, and the Starnet project file **(.SNPROJ)**.

Important Note:

Once a network has been adjusted and coordinates are reported to another entity (e.g.: Mapping Unit), these coordinates shall be deemed final. Should supplemental control or boundary ties be needed, the primary coordinates shall be fixed in subsequent adjustments. Only in the event that erroneous data is discovered will previously reported coordinate values be changed.

Appendix A – Formulas

1. Minimum spacing for new control points to be positioned using GNSS can be computed using the following formula:

D = 10,000 x √ (2 x { [(1.96)(a)]2 + [(1.96)(b)]2 + c2 })
where:

- D = minimum spacing (in feet) between static or RTK occupation stations
- a = manufacturer's millimeter rating at a 68% confidence level, (converted to feet)
- b = manufacturer's ppm rating at a 68% confidence level, times the average distance (in feet) from legacy control stations, and divided by 1,000,000
- c = estimated receiver positioning error (rod plumb or tribrach errors), commonly estimated to be 0.007 feet
- 1.96 = the multiplier from a 68% confidence level (1 sigma) to a 95% confidence level (2 sigma)

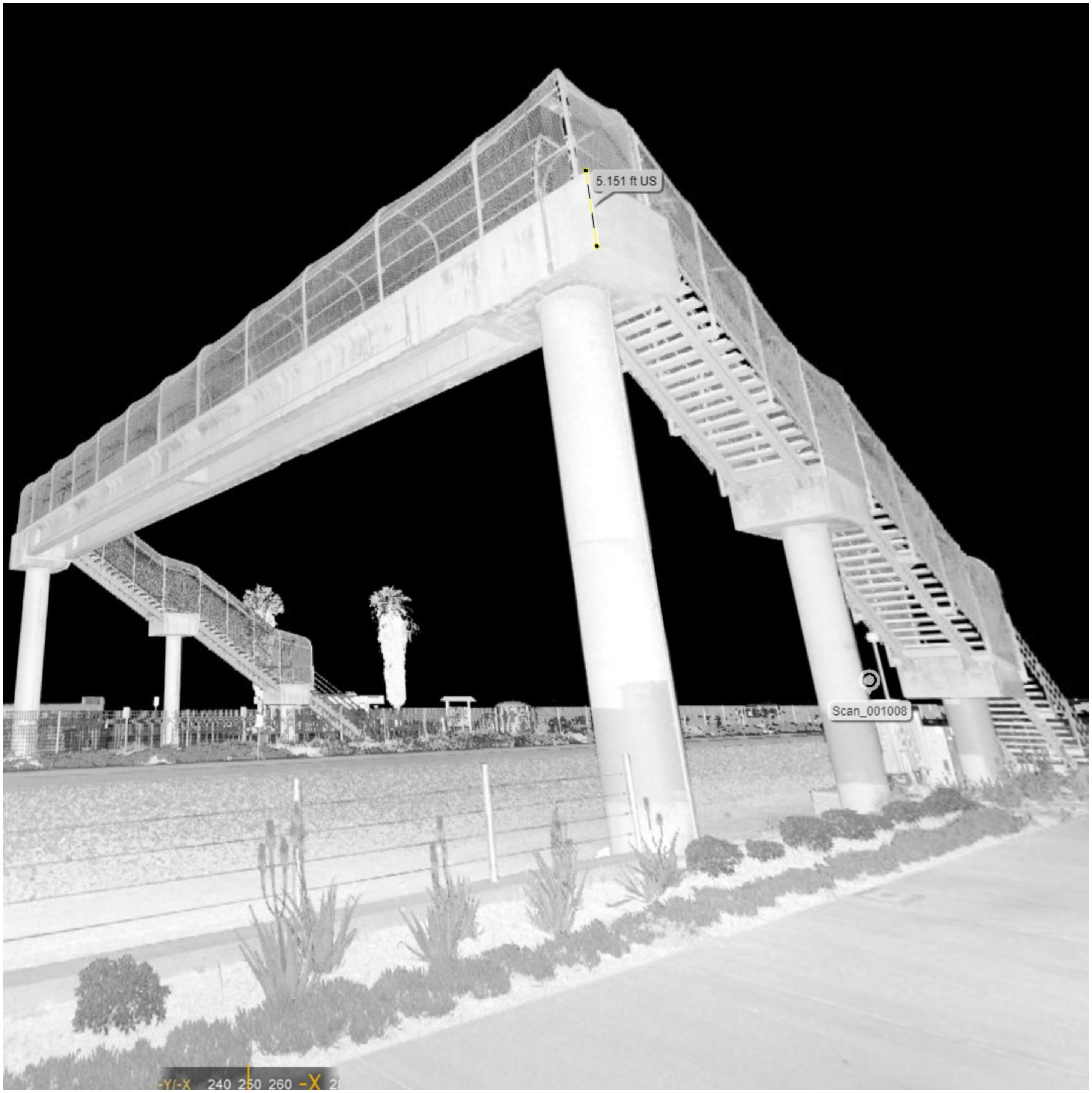
2. All connected monument pairs shall pass the following mathematical test:

D ÷ √ (x2 + y2) ≥ 10,000 (or ≥ 20,000 where required above)
where:
- D = distance (in feet) between the pair of monuments being examined
- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

3. Connections of very short distances often will not meet the **1:10,000** standard defined by the formula in Section 2 above. An alternative standard for distances of less than 330 feet follows:

√ (x2 + y2) ≤ 0.033 feet
where:
- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

CHAPTER 7
LIDAR SURVEYS



Policy Statement

Any survey which incorporates LiDAR data shall conform to the specifications as defined in this document.

General Statement

The term "LiDAR" is short for Light Detection and Ranging and is also commonly referred to as "Laser Scanning". A LiDAR survey can be performed from many platforms, including fixed-wing aircraft, Unmanned Aerial Systems (UAS, "drones"), and ground based systems (stationary or mobile). The purpose of this document is to define standards for the use of ground based LiDAR only. See Chapter 10 – Small Unmanned Aerial Vehicle (sUAV) Surveys for alternate methods of LiDAR data collection.

Applications for LiDAR

LiDAR is an efficient tool for the following types of surveys:

- Engineering topographic surveys including roadways, channels, and structures
- As-built surveys
- Determination of vertical clearances (bridges, overhead power lines, etc.)
- Deformation monitoring
- Volumetric Surveys
- Architectural or archaeological surveys

Benefits of LiDAR

The benefits of a LiDAR survey as compared with conventional methods of data collection are as follows:

- Unsafe conditions inherent to high traffic zones or other hazardous environments may be mitigated
- Massive amounts of data can be collected in a very short period of time
- The same data set may be used for multiple projects with multiple delivery formats
- Go-backs are virtually eliminated

Challenges Inherent to LiDAR

LiDAR is not the perfect tool for all applications. Following are some of the challenges inherent to a LiDAR survey:

- Office processing time is increased substantially as compared to conventionally collected data sets

- Control network must be of a higher density than that of conventional data collection methods
- Equipment currently available barely meets required accuracy standards; extra care must be taken to create ideal conditions for data capture, e.g.: densification of the control network, shortening sighting distances, maximizing incidence angles, increasing overlap, etc.
- Features obscured by dirt or foliage are essentially invisible to the scan and must be collected conventionally
- Laser equipment presents a potential for eye injury; operators of laser equipment must follow OSHA Regulation 1926.54 entitled "Nonionizing Radiation"

Preparation

Before planning the survey, a meeting between the Party Chief and/or Senior Land Surveyor and the Requestor shall be conducted on the job site. During this meeting, the Requestor will explain specific project requirements as detailed on the Initial Survey Request. It is the responsibility of the Party Chief to listen attentively and take notes, and to bring up any questions or concerns. If any ***significant*** changes to the scope of the project are identified and agreed upon during this meeting, the Requestor must complete an Additional Survey Request.

After this meeting, if it has been determined that the project will indeed incorporate LiDAR data, a Project Work Plan shall be drafted. The Work Plan establishes:

- Required accuracy level to be met
- Type of equipment to be used
- Spacing and approximate locations of scan targets and validation points
- Identification of potential obscured areas
- QA/QC plan
- Detailed list of deliverables

Each of these items will be reviewed in greater detail below.

Accuracy Requirements

Regardless of the compilation methods used, topographic maps are expected to conform to the FGDC Geospatial Positioning Accuracy Standards, Part 4: Standards for A/E/C and Facility Management, which defines requirements for quality control testing, and which references the ASPRS Positional Accuracy Standards for Digital Geospatial Data, currently in Edition 1, Version 1.0, November 2014.

Both of these standards define positional tolerances assessed for horizontal and vertical components separately. Accuracy assessment is to be achieved by sampling the mapping with validation points from a higher level of accuracy if possible. It is also possible to claim an accuracy level by following practices that have been otherwise proven. LiDAR mapping must also include the Nominal Pulse Density - NPD or Nominal Pulse Spacing - NPS as a measure of resolution.

The vertical component is assessed separately between non-vegetated areas (Non -Vegetated Vertical Accuracy - NVA) and vegetated areas (Vegetated Vertical Accuracy - VVA). The assessments are relative to the established ground control, and are listed as Root Mean Square Error - RMSE (the 67% confidence level), and at 2 Standard Deviations (the 95% confidence level). The following tables were compiled in summary of the ASPRS and FGDC standards. ASPRS and National Map Accuracy Standards legacy mapping accuracy standards are listed in approximate equivalency for reference:

Horizontal Accuracy / Quality Examples for High Accuracy Digital Planimetric Data

ASPRS 2014			Equivalent Map Scale	
Horizontal Accuracy Class RMSE X,Y (cm)	RMSE - X & Y Combined (cm)	Horizontal Accuracy at 95% Confidence (cm)	ASPRS 1990 - Class 1	NMAS 1943
0.63	0.9	1.5	1:25	1:16
1.25	1.8	3.1	1:50	1:32
5	7.1	12.2	1:200	1:127
7.5	10.6	18.4	1:300	1:190
10	14.1	24.5	1:400	1:253
12.5	17.7	30.6	1:500	1:317
15	21.2	36.7	1:600	1:380
30	42.4	73.4	1:1200	1:760
60	84.9	146.9	1:2400	1:1521
150	212.1	367.2	1:6000	1:3802

Vertical Accuracy / Quality Examples for Digital Elevation Data; Recommended LiDAR Nominal Pulse Spacing (NPS); Equivalent Contour Interval

ASPRS 2014			Equivalent Contour Interval	
Vertical Accuracy Class - RMSE Z NVA (cm)	Absolute Accuracy NVA at 95% Confidence (cm)	Recommended Maximum NPS (meters)	ASPRS 1990 - Class 1 (cm)	NMAS 1943 (cm)
1	2.0	< 0.22	3.0	3.29
2.5	4.9	0.25	7.5	8.22
5	9.8	0.35	15.0	16.45
10	19.6	0.71	30.	32.90
15	29.4	1.0	45.0	49.35
20	39.2	1.4	60.0	65.80
33.3	65.3	2.0	99.9	109.55
66.7	130.7	3.2	200.1	219.43
100	196.0	4.5	300.0	328.98
333.3	653.3	10.0	999.9	1096.49

Based upon the accuracy requirements defined above and the ultimate purpose of the survey, a determination must be made as to the density and distance limits of the scan. A ***"Fine Scan"*** should result in an NPS of ≤ 25mm at 50 meters. A ***"Coarse Scan"*** should result in an NPS of ≤ 50mm at 50 meters. When capturing hardscape features, a Fine Scan is required, and scan distances shall be limited to 150 feet. When capturing utility appurtenances, original ground features, landscaping features, or for volumetric surveys, a Coarse Scan may be used, and scan distances shall be limited to 300 feet. When equipment specifications vary from those stated above, scan distance limits shall be defined by manufacturer's specifications, in order to obtain the desired NPS.

If a specific sector within a project presents field conditions which preclude achievement of the desired project accuracy level (e.g. dense vegetation), the sector is to be segregated within an appropriately-identified polygon.

Stationary Scans

A stationary scan is one in which the scanning instrument is mounted on a fixed platform, typically a tripod. Field procedures are outlined below:

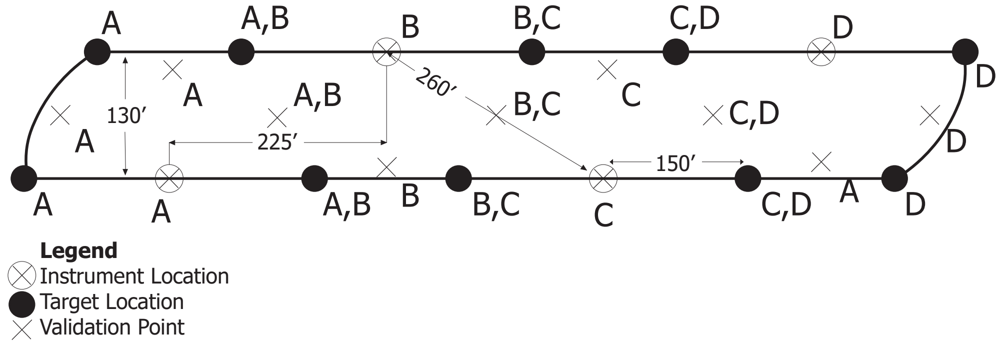
Instrument Positioning and Control Target/Validation Point Layout:

Instrument positioning and control target/validation point layout will vary depending on the type of equipment selected and the desired density of the scan.

Details are provided below:

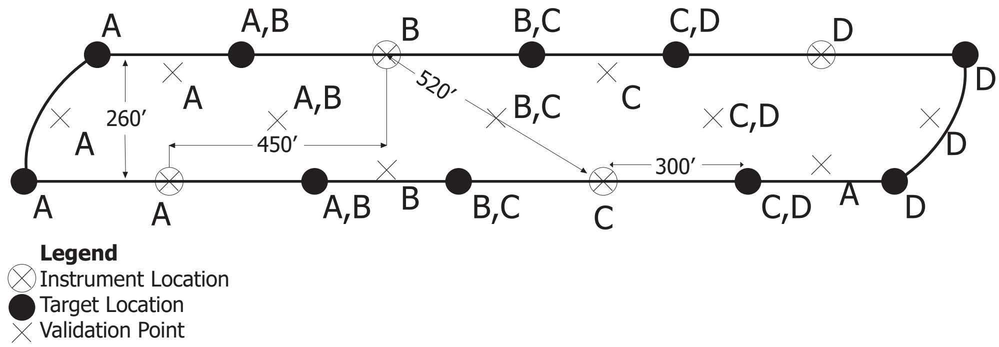
- The sections below entitled "Establishing Horizontal Control", "Establishing Vertical Control", and "Establishing Scan Control Targets and Validation Points", provide detailed requirements for the establishment of project control and positions of control targets and validation points.
- The Senior Land Surveyor shall approve the control scheme and target layout prior to execution of the scan.
- The scanning instrument shall be positioned so as to achieve a 15% overlap with adjacent scans.
- Refer to Figure 1a and Figure 1b below for typical distribution of instrument, target, and validation points.
- If the scanning instrument cannot be set up over a known control point and oriented towards a known backsight point, the instrument position shall be determined by resection from a minimum of four target points.
- If the scanning instrument can be set up over a known control point and oriented towards a known backsight point, the additional target points shown in Figure 1a and Figure 1b may be eliminated, and a third known control point shall be staked-out and stored before execution of the scan.
- Control targets and validation points shall span the entire scanning view scene, both horizontally and vertically.
- Depending upon the type of equipment used for the scan, control target points shall be occupied by either a conventional prism assembly or a spherical/planar type target designed specifically for LiDAR scans.
- Some processing software is capable of using vertical planes (e.g. signs, walls) as scan control. When it is possible to do so, it is beneficial to the registration process to locate control targets at or near such features.
- Validation points may consist of painted or pre-made targets, or clearly discernable existing features such as the intersection of traffic stripes, an angle point in a concrete feature, etc.
- A minimum of three validation points shall be included in each scan.

Figure 1a - Target Layout for Fine Scan



Note - "Fine Scan" is defined by point spacing at 50m < 25mm

Figure 1b - Target Layout for Course Scan



Note - "Course Scan" is defined by point spacing at 50m < 50mm

Conducting the Scan:

The following notes relate to the workflow of stationary scans:

- Calibration of the scanning instrument shall be performed daily throughout the duration of the project, or as specified by the manufacturer.
- When capturing non-vertical features, the scanning instrument should be set up as high as is practical, so as to maximize the angle of incidence with reflective surfaces.
- Instrument set-up information shall be logged on the relevant setup sheet ("LiDAR Set-Up Sheet – Known Point" or "LiDAR Set-Up Sheet – Random Position"), including a sketch of the control targets and validation points to be scanned.
- Instrument and target heights shall be measured and recorded in both metric and imperial units, with a unit conversion calculation performed before collection of data.
- If possible, a maximum scanning distance should be assigned in order to avoid capturing data beyond desired limits.
- Control targets are scanned at maximum density and the scan is registered to project control, either by resection or by occupation of known control.
- The remainder of the scan is now conducted at the desired density.
- The point cloud shall be reviewed before the scanner is moved, and any features which may not have been accurately captured shall be described on the set-up sheet. These features must later be collected by an alternate method.

Mobile Scans

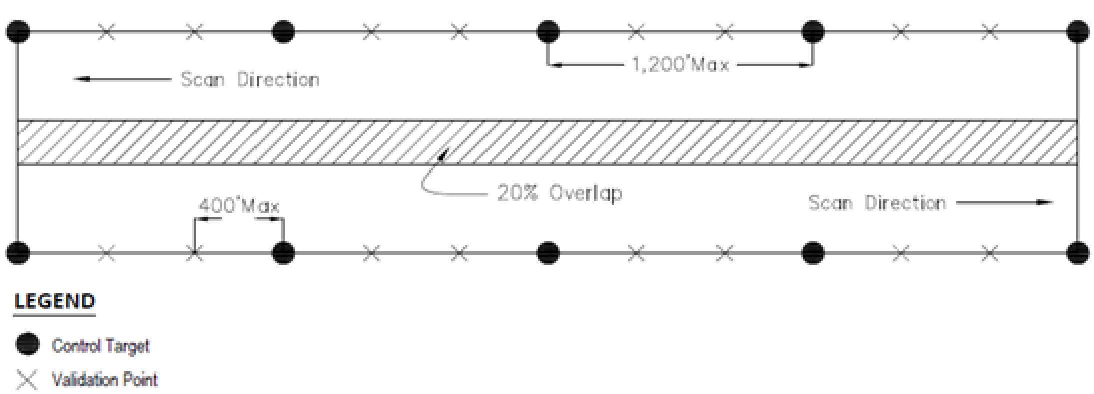
A mobile scan is one in which one or more scanning instruments are mounted on a mobile platform, such as a motor vehicle. The scanning instrument collects data continuously as the position of the vehicle is determined by a combination of control targets, GNSS receivers, and an Inertial Measurement Unit (IMU). Field procedures are outlined below:

Scan Control Target/Validation Point Layout:

Control target and validation point layout will vary depending upon existing field conditions. Details are provided below:

- The sections below entitled "Establishing Horizontal Control", "Establishing Vertical Control", and "Establishing Scan Control Targets and Validation Points", provide detailed requirements for the establishment of project control and positions of scan control targets and validation points.
- The Senior Land Surveyor shall approve the control scheme and control target/ validation point layout prior to execution of the scan.
- Control targets shall consist of painted or pre-made targets with a distinct and recoverable survey point, while validation points may consist of targets or clearly discernable existing features, such as the intersection of traffic stripes, an angle point in a concrete feature, etc.
- Refer to **Figure 2** below for typical distribution of control targets and validation points.
- Some processing software is capable of using vertical planes (e.g. signs, walls) as scan control. When it is possible to do so, it is beneficial to the registration process to locate control targets at or near such features.
- Spacing of control targets and validation points should be decreased in areas where GNSS signal may be lost or degraded due to tree canopy, steep embankments, overpasses, etc.

Figure 2 - Target Layout for Mobile Scans



GNSS Requirements:
Following are requirements for collection and processing of GNSS data:

- GNSS data is collected with a conventional base/rover configuration, with base receivers located within the project area and rover receiver(s) mounted on the vehicle.
- GNSS base stations shall be located at a maximum of 6 mile intervals in order to ensure that no baselines over 3 miles in length are used in the solution. Projects 6 miles or less in length should have a base station at each end of the project.
- Due to the 3 mile maximum baseline length requirement, CRTN CGPS stations are typically not to be used as GNSS base stations.
- Selection of GNSS base station locations should be based on actual field conditions, avoiding nearby trees, overhead powerlines, and other obstructions or sources of signal noise and multipath.
- GNSS receivers shall be a minimum of dual frequency, and data shall be collected at 0.5 second or 1 second intervals.
- Satellite visibility and PDOP forecasts shall be reviewed prior to scheduling the scan: a minimum of 6 satellites common to the base and rover receivers should be observed and PDOP should not exceed 3.0.
- Maximum duration of GNSS signal loss or degradation is 60 seconds or 0.6 miles travelled, during which time the IMU must be operational within the following parameters: maximum uncorrected horizontal drift error of 0.33 feet; maximum uncorrected vertical drift error of 0.23 feet; maximum uncorrected roll and pitch error of 0.020 degrees RMS; maximum uncorrected true heading error of 0.020 degrees RMS (this data was copied from Chapter 15 of the Caltrans Surveys Manual).
- Kinematic GNSS data shall be post processed in both forward and reverse directions.

Inertial Measurement Unit (IMU) Requirements:
Following are requirements for the IMU associated with the scanning project (this data was copied from Chapter 15 of the Caltrans Surveys Manual). Note that rapidly changing technology as related to this equipment may result in modification of these current requirements:
- Minimum positioning data sampling rate of 100 Hz
- Maximum gyro rate bias of 1 degree per hour
- Maximum gyro rate scale factor of 150 ppm
- Maximum Angular Random Walk of 0.125 degree per √hour
- Minimum uncorrected positioning capability due to loss or degradation of GNSS signal of 60 seconds or 0.6 miles

Conducting the Scan:
The following notes relate to the workflow of mobile scans:

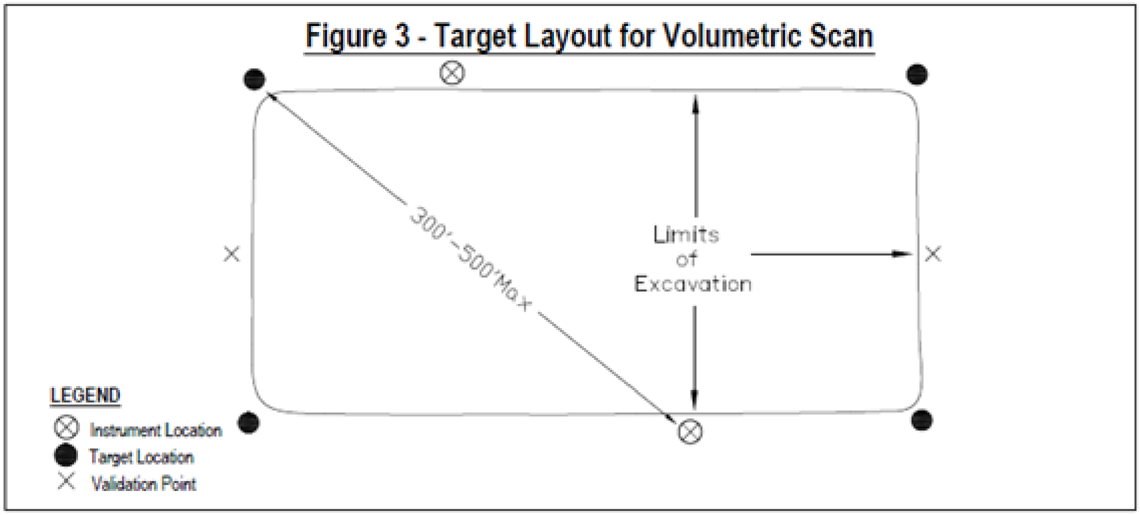
- Calibration of the scanning instrument and IMU shall be performed daily throughout the duration of the project, or as specified by the manufacturer.
- GNSS receivers should be set up and collecting data at all base station locations throughout the scanning operation.
- GNSS signal shall be monitored continuously during the scanning operation, and the duration of any loss or degraded signal shall be documented for verification of compliance with this document.
- The IMU system must be monitored continuously during the scanning operation.
- Vehicle speed is to be adjusted so as to ensure desired data point density is achieved.
- Multiple scans shall be collected - in both directions or with two passes in the same direction - with a planned side overlap (sidelap) of 20%.

Volumetric Scans

A volumetric scan is one which facilitates calculation of earthwork volumes, e.g.: stockpiles, keyway removal areas, etc. Although volumetric surveys are often conducted by UAV (see Chapter 10 – Small Unmanned Aircraft System (sUAS) Surveys), the purpose of this section is to define standards for the use of ground based scans only. Field procedures are outlined below:

Instrument Positioning and Control Target/Validation Point Layout:
The same basic standards defined above for stationary scans apply to volumetric scans. Variations from these standards and further explanation are defined below:

- Volumetric scans may be captured at a low point density (course scans).
- Scan distances should be limited to 300 feet but may be extended to 500 feet at the discretion of the party chief, with approval from the Senior Land Surveyor.
- Regardless of the size of the stockpile or excavation, the scanning instrument shall be set up in at least two locations in order to capture all faces of the earthwork and to provide the required overlap between scans.
- A minimum of two validation points shall be included in each scan.
- Elevations of instrument, control target, and validation points may be derived by trigonometric principles.
- Refer to **Figure 3** on the next page for typical distribution of control target and validation points.



Quality Assurance/Quality Control Report

Prior to commencement of field activities, a Quality Assurance/Quality Control (QA/QC) plan, developed by the Senior Land Surveyor, must be in place. Once the field survey and data processing have been completed, the QA/QC Report is prepared, which shall include the following:

- Listing of all equipment used on the project
- Statistical summary of the control network (Star*Net adjustment)
- Accuracy validation methodology, including registration statistics, control target and validation point comparison statistics, scan seam comparison of overlap areas, etc.
- Log of PDOP values (mobile scan)
- Log of IMU statistics (mobile scan)
- Separation between the results of forward and reverse processing of kinematic GNSS data (mobile scan)

Deliverables – Consultant to OC Survey

A consultant performing a LiDAR survey for OC Survey shall provide the following list of deliverables:

- QA/QC Report
- Project control schematic, including the associated Star*Net adjustment report
- Differential leveling notes
- LiDAR set-up sheets, field sketches, etc.
- Registered point cloud data in XYZI (or XYZIRGB) format
- Image overlay data
- Civil 3D files

The format of each of these items shall be agreed upon by all parties prior to commencement of scanning operations.

Deliverables – OC Survey to OC Design

A list of deliverables and formats shall be defined on the Initial Survey Request form received from the Requestor and should be verified during the meeting discussed above in the section entitled "Preparation". Typical deliverables are listed below:

- Registered point cloud data in XYZI (or XYZIRGB) format
- Image overlay data
- Civil 3D files

Establishing Horizontal Control

Computed positions of control points shall conform to a minimum combined (relative) positional accuracy* of **1:10,000** (at a 95% confidence level, or 2 sigma), or a combined distance error of **≤ 0.033** feet for connection distances shorter than **330 feet**. This relative positional accuracy standard shall be met whether the survey is conducted by GNSS (static or RTK), conventional traverse (total station), or any combination thereof. Positions of base stations used in mobile LiDAR applications shall conform to a minimum combined (relative) positional accuracy of **1:20,000**. In addition, each base station shall have a computed network accuracy (error ellipse semi-major axis) **≤ 0.033 feet**.

*Relative positional accuracy is a measure of the accuracy of point positions in relation to each other, and is not to be confused with the measure of traverse closure expressed as a ratio.

The following are guidelines for GNSS and conventional traverse methodology:

Static GNSS:

Control for a LiDAR survey project may be established by static (or fast-static) GNSS procedures. While a network adjustment may be performed using only GNSS vectors (stand-alone), combining conventional traverse data with GNSS vectors will sometimes result in a network with higher relative positional accuracy.

Design of the network and occupation scheme will be determined by the Party Chief in conformance with Chapter 1 – Static GNSS When selecting points to be included in the static network, consideration must be given as to strength of figure and adequate spacing. The minimum allowable spacing for points in stand-alone networks shall be dictated by the following criteria:

- Trimble R10 receivers, rated for static surveys at 3mm + 0.5 ppm at 68% confidence level (1 sigma): a minimum spacing of **500 feet** when tied to CGPS stations at an average distance of 32,000 feet, and a minimum spacing of **300 feet** when tied to primary project control or legacy control at an average distance of 4,000 feet
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

RTK GNSS:

RTK is generally not to be used as a stand-alone measurement tool when performing a control survey. RTK is best used to bolster the control network, not define it. In order to ensure realization of the 1:10,000 criteria, the network shall be adjusted using RTK measurements together with conventional traverse data.

RTK occupation points are selected in such a way as to maximize strength of figure, while leaving the bulk of the data to be captured by conventional traverse. The occupation scheme will be determined by the Party Chief in conformance with Chapter 2 – RTK GNSS. The minimum recommended spacing for points in RTK surveys shall be dictated by the following criteria:

- Trimble R10 receivers, rated for RTK surveys at 8mm + 1 ppm at 68% confidence level (1 sigma): a minimum spacing of 1200 feet when tied to CRTN stations at an average distance of 32,000 feet; a minimum spacing of 700 feet when tied to local project control in a base-rover configuration at an average distance of 4,000 feet.
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1".

Conventional Traverse (Total Station):

Conventional traversing may be used either as a stand-alone method or in combination with GNSS vectors when establishing control networks for LiDAR surveys.

Field measurements made with a total station shall meet the following specifications:

- Horizontal Angles: Minimum of two direct (face 1) and two reverse (face 2) with a maximum residual of 5 seconds; residual exception granted for sights closer than 300 feet
- Distances: Measured to backsight and foresight; minimum of two direct and two reverse with a maximum residual of 0.007 feet

Establishing Vertical Control

For LiDAR surveys representing hardscape features (concrete, asphalt), elevations of control , target, and validation points shall be established using differential leveling procedures, meeting 3rd order (or better) accuracy requirements. Leveling shall be referenced to a minimum of two vertical control points (benchmarks) and be in conformance with Chapter 4 – Differential Leveling. For LiDAR surveys representing original ground features, landscaping features, or for volumetric surveys, elevations of control, target, and validation points may be derived by trigonometric principles.

Establishing Scan Control Targets and Validation Points

Scan control targets may be incorporated into the master project control scheme, or they may be positioned from project control as a secondary process. In either case, computed positions of control targets shall conform to the same minimum combined (relative) positional accuracy and procedural requirements as the master project control scheme.
Validation points may be located from project control points or scan control target points as "sideshots".

Adjustment of the Control Network

All LiDAR control data shall be adjusted by least squares adjustment software, in conformance with Chapter 12 – Network Processing.

Statistical analysis of the network adjustment shall be performed to ensure that a minimum combined (relative) positional accuracy of 1:10,000 (1:20,000 for GNSS base station points used for mobile scans) has been achieved for all connected monument pairs. Although this computation is automatically performed in most network adjustment software, the formula for this computation is shown in "Appendix A, Section 2".
Connections of very short distances often will not meet this 1:10,000 standard. An alternative standard for distances of less than 330 feet is shown in "Appendix A, Section 3".

In the event one or more pairs of monuments fail to pass these relative positional accuracy criteria, the network adjustment shall be reviewed and a determination made by the Senior Land Surveyor (or Project Manager) as to whether or not additional observations will made in order to improve geometry, increase redundancy, or further isolate errors.

Important Note:

Once a network has been adjusted and coordinates are reported to another entity (e.g.: Mapping Unit), these coordinates shall be deemed final. Should supplemental control or boundary ties be needed, the primary coordinates shall be fixed in subsequent adjustments. Only in the event that erroneous data is discovered will previously reported coordinate values be changed.

Monumentation

Monuments set as control points during the course of a LiDAR survey shall meet the following criteria:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of an iron pipe with a tag or disk, or a rebar with an aluminum cap. Rebar must be set a minimum of 3 inches below the ground surface.
- All tags/washers/disks/caps referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge, and shall also be stamped "CP" or "CONTROL POINT".
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe or rebar.

Additional Resources:

Chapter 15 of the Caltrans Surveys Manual is a valuable resource which should be consulted before planning a LiDAR survey.

Appendix A – Formulas

1. Minimum spacing for new control points to be positioned using GNSS can be computed using the following formula:

$$D = 10{,}000 \text{ x } \sqrt{}\,(2 \text{ x } \{ [(1.96)(a)]2 + [(1.96)(b)]2 + c2 \})$$

where:

- D = minimum spacing (in feet) between static or RTK occupation stations
- a = manufacturer's millimeter rating at a 68% confidence level, (converted to feet)
- b = manufacturer's ppm rating at a 68% confidence level, times the average distance (in feet) from legacy control stations, and divided by 1,000,000
- c = estimated receiver positioning error (rod plumb or tribrach errors), commonly estimated to be 0.007 feet
- 1.96 = the multiplier from a 68% confidence level (1 sigma) to a 95% confidence level (2 sigma)

2. All connected monument pairs shall pass the following mathematical test:

$D \div \sqrt{}\,(x2 + y2) \geq 10{,}000$ (or $\geq 20{,}000$ where required above)

where:

- D = distance (in feet) between the pair of monuments being examined
- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

3. Connections of very short distances often will not meet the 1:10,000 standard defined by the formula in Section 2 above. An alternative standard for distances of less than 330 feet follows:

$\sqrt{}\,(x2 + y2) \leq 0.033$ **feet**

where:

- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

CHAPTER 8
SCOUR STUDY SURVEYS

Policy Statement

Any survey performed for the purpose of collecting data for Scour Studies shall conform to the specifications as defined in this document.

Objective

The objective of a Scour Study survey is to determine the degree of sedimentation or scour (erosion) which has occurred within flood control facilities. Data is collected yearly or as directed by the requestor, typically OC Operations and Maintenance (O&M). Data points are usually collected in the form of cross sections at pre-determined stations. Volumes are computed by comparing cross sections to those from the previous survey, and a comparison is also made with the cross section templates from the original as-built drawings.

Accuracy Requirements

Data points should be located within the following tolerances, relative to project control ("local accuracy"):

- Data points should be located within +/- 0.10 feet horizontally and vertically.
- Data points should ideally be located within +/- 1 foot of the desired pre-determined station, with an occasional variance of up to +/- 5 feet accepted at the discretion of the Senior Land Surveyor

Field Methodology

This document establishes acceptable methodology for Scour Study surveys incorporating conventional instruments (total stations) and RTK GNSS. See Chapter 9 – Hydrographic Surveys and Chapter 10 – Small Unmanned Aerial Vehicle (sUAV) Surveys for alternate methods of data collection.

See "Establishing the Control Network" below for details on the horizontal and vertical control network upon which the Scour Study survey is based.

General Methodology:

The following general notes relate to data collected from any source:

- Instrument set-up information, point ranges used, edits needed, etc. shall be logged on a "Data Collection Set-Up Sheet".

- Spacing of data points shall be such as is necessary to capture an accurate cross-sectional representation of the horizontal and vertical geometry of the flood control facility. Determination of point spacing is made in the field by the Party Chief.
- When capturing data points within any soft surface, e.g. sand or mud, the layout rod shall be equipped with a "shoe" or other blunt object designed to keep the rod from sinking beneath the surface.

Conventional Instruments:

The following notes are specific to data collected with a conventional instrument:

- After the instrument is oriented and the measurement to the backsight is recorded, a third control point is staked out, read, and recorded using the layout rod that will be used for collection of data points. This provides assurance that the prism offset and rod HI measurement are correct. Check shots should be coded with a unique numbering system which makes them easy to sort and verify. For example, a check shot on point #207 could be named "CHK207".
- In order to prevent degradation of the horizontal and vertical accuracy of data points, measurement distances should be limited to a maximum of **800 feet**.
- Prism rod HI may be extended to **25 feet**.

RTK GNSS:

The following notes are specific to data collected with RTK GNSS (RTK):

- Refer to Chapter 2 - RTK GNSS for detailed policy on the use of RTK.
- For RTK surveys based on the California Real Time Network (CRTN), a 3D Site Calibration is required (see "Appendix B" for Site Calibration procedure). RTK surveys based on existing project control using a Base/Rover configuration do not require Site Calibration, provided that Base/Rover procedures defined in **Chapter 2** are followed.
- A known point shall be checked at the beginning and end of each session. Prudent practice would indicate additional checks, particularly after initialization is lost. Check shots should be coded with a unique numbering system which makes them easy to sort and verify. For example, a check shot on point #207 could be named "CHK207".
- Each topographic data point shall consist of a single measurement of at least **5 epochs**. Measurement duration of data points adjacent to steep embankments or tree canopy, or under any other circumstance which results in decreased precision, shall be prolonged so as to ensure the desired accuracy has been met.

Monumentation

Monuments set as control points during the course of a scours study survey shall meet the following criteria:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of an iron pipe with a tag or disk, or a rebar with an aluminum cap. Rebar must be set a minimum of 3 inches below the ground surface.
- All tags/washers/disks/caps referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge, and shall also be stamped "CP" or "CONTROL POINT".
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe or rebar.

Office Workflow

The following office workflow is to take place as the fieldwork progresses:

- Field data should be downloaded at the end of each day. Data shall be reviewed and edited by or under the supervision of the Party Chief. Care must be taken to resolve errors in rod heights and incomplete or inaccurate cross section lines.
- This edited data shall be delivered to the Mapping Unit (placed on the Field Survey Server) in blocks of no larger than three days of fieldwork.
- Note that the files themselves are not delivered by email. Files shall be copied to the Field Survey Server and organized as described in Chapter 13 – Preparation of the Field Note Package. A link to the file location is emailed to the Mapping Unit.
- Data is delivered in Starnet **DAT** format (see "Adjustment of the Network" below for more details on generating the **DAT file**). The Starnet project file (**.SNPROJ**) containing all project settings is also delivered, and as new field data is collected, the Mapping Unit and the Party Chief will each maintain a running amended adjustment as an additional layer of QA/QC.
- Along with the Starnet data, the Party Chief shall deliver data collector job files (.JOB) and copies of the Data Collection Set-Up Sheets.

- After completion of the fieldwork and topo processing, the Mapping Unit will provide the Party Chief with a cross section plot of the entire project. The Party Chief shall walk the jobsite with the plot in hand, searching for errors or omissions, making corrections as needed. At the discretion of the Senior Land Surveyor, in lieu of a site visit the Party Chief may perform an office review of cross section plots.
- After final edits have been made, a plot is provided for the Senior Land Surveyor to review. Cross sections are delivered to the requestor in plot and tabular format, with quantities of cut or fill for each station and for the project as a whole.

Establishing the Control Network

In most cases, Scour Study surveys are performed using a control network established by a previous survey. If existing control is to be used as the basis of the survey, the integrity of the network shall be verified before the survey commences. This verification process must be documented in the form of a narrative, listing of stakeout deltas, and/or StarNet report. Any additional control points needed are then tied to this existing control and included within the project deliverables.

In the case of a new facility or a facility in which a substantial percentage of the original control has been destroyed, establishment of the new control network shall follow the procedures outlined below:

Horizontal control:

Horizontal control for Scour Study surveys shall conform to a minimum combined (relative) positional accuracy of **1:10,000** (at a 95% confidence level, or 2 sigma), or a combined distance error of **≤ 0.033** feet for connection distances shorter than **330 feet**. Relative positional accuracy is a measure of the accuracy of point positions in relation to each other, and is not to be confused with the measure of traverse closure expressed as a ratio.

This **1:10,000** standard shall be met whether the survey is conducted by GNSS (static or RTK), conventional traverse (total station), or any combination thereof.

The following are guidelines for GNSS, conventional traverse, and differential leveling methodology:

Static GNSS:
Control for a Scour Study survey may be established by static (or fast-static) GNSS procedures. While a network adjustment may be performed using only GNSS vectors (stand-alone), combining conventional traverse data with GNSS vectors will result in a network with higher relative positional accuracy.

Design of the network and occupation scheme will be determined by the Party Chief in conformance with Chapter 1 – Static GNSS When selecting points to be included in the static network, consideration must be given as to strength of figure and adequate spacing. The minimum allowable spacing for points in stand-alone networks shall be dictated by the following criteria:

- Trimble R10 receivers, rated for static surveys at 3mm + 0.5 ppm at 68% confidence level (1 sigma): a minimum spacing of **500 feet** when tied to CGPS stations at an average distance of 32,000 feet, and a minimum spacing of **300 feet** when tied to primary project control or legacy control at an average distance of 4,000 feet
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

RTK GNSS:
RTK is generally not to be used as a stand-alone measurement tool when performing a control survey. RTK is best used to bolster the control network, not define it. In order to ensure realization of the **1:10,000** criteria, the network shall be adjusted using RTK measurements together with conventional traverse data.

RTK occupation points are selected in such a way as to maximize strength of figure, while leaving the bulk of the data to be captured by conventional traverse. The occupation scheme will be determined by the Party Chief in conformance with Chapter 2 – RTK GNSS.

The minimum recommended spacing for points in RTK surveys shall be dictated by the following criteria:

- Trimble R10 receivers, rated for RTK surveys at 8mm + 1 ppm at 68% confidence level (1 sigma): a minimum spacing of **1200 feet** when tied to CRTN stations at an average distance of 32,000 feet; a minimum spacing of **700 feet** when tied to local project control in a base-rover configuration at an average distance of 4,000 feet
- Minimum spacing for GNSS receivers with static survey ratings different from those listed above can be computed using the formula shown in "Appendix A, Section 1"

Conventional Traverse (Total Station):
Conventional traversing may be used either as a stand-alone method or in combination with GNSS vectors when establishing control networks for Scour Study surveys.

Field measurements shall meet the following specifications:

- Horizontal Angles: Minimum of two direct (face 1) and two reverse (face 2) with a maximum residual of 5 seconds; exception granted for sights closer than 300 feet.
- Distances: Measured to backsight and foresight; minimum of two direct and two reverse with a maximum residual of 0.007 feet.

Vertical Control:
For Scour Study surveys, elevations of points within the primary control network shall be established using differential leveling procedures. Leveling shall be referenced to a minimum of two vertical control points (benchmarks) and be in conformance with Chapter 4 – Differential Leveling.

Elevations of subsequent supplemental control may be derived by trigonometric principles, provided the points are traversed through, double determined, or set by two-point resection, with acceptable mathematical vertical closures observed.

Adjustment of the Network

Control Network:

All GNSS and conventional data shall be adjusted by least squares adjustment software in conformance with Chapter 12 – Network Processing.

Statistical analysis of the adjustment shall be performed to ensure that a minimum combined (relative) positional accuracy of **1:10,000** has been achieved for all connected monument pairs. Although this computation is automatically performed in most network adjustment software, the formula for this computation is shown in "Appendix A, Section 2".

Connections of very short distances often will not meet this **1:10,000** standard. An alternative standard for distances of less than **330 feet** is shown in "Appendix A, Section 3".

In the event one or more pairs of monuments fail to pass these relative positional accuracy criteria, the network adjustment shall be reviewed and a determination made by the Senior Land Surveyor (or Project Manager) as to whether or not additional observations will made in order to improve geometry, increase redundancy, or further isolate errors.

Topo Sideshots:

After the control network has been satisfactorily adjusted, Scour Study data points are added. Unique **DAT files** for each block of data (data representing 1 to 3 days of fieldwork) are created from the data collector files. Be sure to export data points in **sideshot** format. All necessary edits are made within the **DAT files.**

All sideshot **DAT files** - original and edited - are delivered to the Mapping Unit, along with **DAT file/s** representing the project horizontal and vertical control network, and the Starnet project file **(.SNPROJ)**.

Important Note:

Once a network has been adjusted and coordinates are reported to another entity (e.g.: Mapping Unit), these coordinates shall be deemed final. Should supplemental control or boundary ties be needed, the primary coordinates shall be fixed in subsequent adjustments. Only in the event that erroneous data is discovered will previously reported coordinate values be changed.

Appendix A – Formulas

1. Minimum spacing for new control points to be positioned using GNSS can be computed using the following formula:

D = 10,000 x $\sqrt{}$ (2 x { [(1.96)(a)]2 + [(1.96)(b)]2 + c2 })

where:

- D = minimum spacing (in feet) between static or RTK occupation stations
- a = manufacturer's millimeter rating at a 68% confidence level, (converted to feet)
- b = manufacturer's ppm rating at a 68% confidence level, times the average distance (in feet) from legacy control stations, and divided by 1,000,000
- c = estimated receiver positioning error (rod plumb or tribrach errors), commonly estimated to be 0.007 feet
- 1.96 = the multiplier from a 68% confidence level (1 sigma) to a 95% confidence level (2 sigma)

2. All connected monument pairs shall pass the following mathematical test:

D ÷ $\sqrt{}$ (x2 + y2) ≥ 10,000 (or ≥ 20,000 where required above)

where:

- D = distance (in feet) between the pair of monuments being examined
- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

3. Connections of very short distances often will not meet the 1:10,000 standard defined by the formula in Section 2 above. An alternative standard for distances of less than 330 feet follows:

$\sqrt{}$ (x2 + y2) ≤ 0.033 feet

where:

- x = error ellipse semi-major axis for monument #1 (at 95% confidence)
- y = error ellipse semi-major axis for monument #2 (at 95% confidence)

Appendix B - Site Calibration Procedure

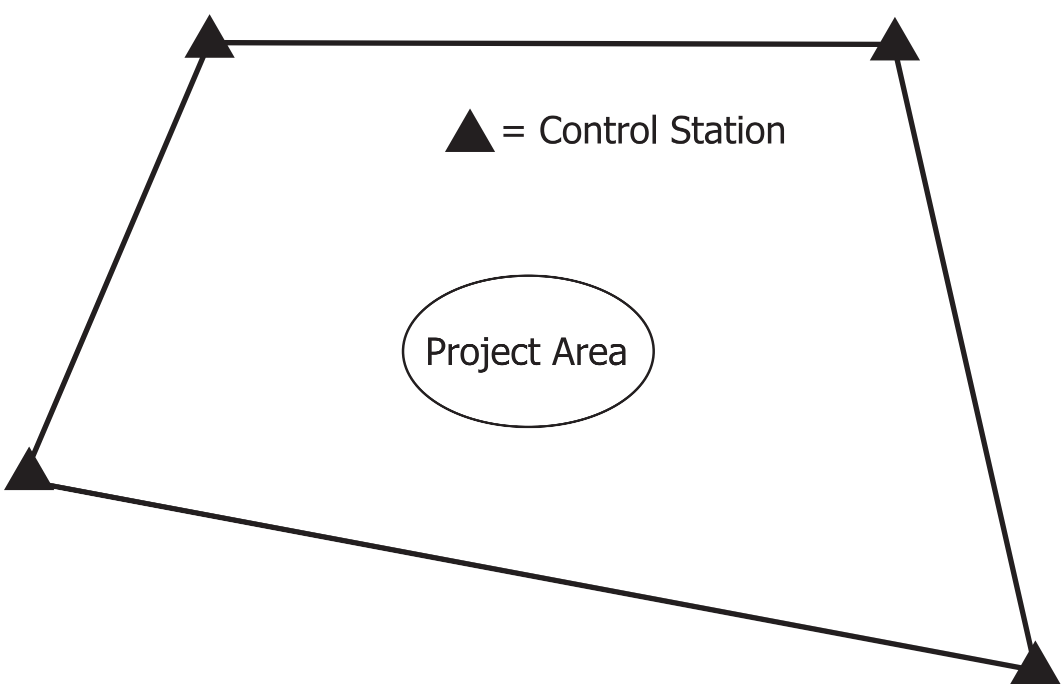
A Site Calibration establishes a relationship between the observed WGS84 coordinates and the local grid coordinates.

The procedures detailed below are specific to topographic survey projects. See Chapter 2 – RTK GNSS for more general uses and procedures for Site Calibrations.

General Requirements:

- The control stations shall be selected so as to create a polygon which fully encompasses the project area (see ***Figure 1***). Selected control stations shall be located no more than six miles from the RTN station or base station.
- Conditions which may generate multipath or obstruct view of the satellites, such as overhead power lines, nearby trees, or adjacent buildings, should be avoided.
- Elevation mask shall be set to 15 degrees.
- Each occupation shall consist of either one measurement of 180 epochs, or three sequential measurements of 60 epochs each.
- Upon computation of the Site Calibration, a control station with residual values greater than those defined below shall be discarded and another control station shall be used in place of this outlier.
- All subsequent measurements and staking activities shall use the same RTN base station or base position as was used to generate the Site Calibration.

Figure 1 – Control Station Selection



2D Site Calibration:

- A minimum of 4 horizontal control stations shall be included in a 2D Site Calibration.
- Each horizontal control station shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 2 hours. These time differentials are required in order to ensure significantly different satellite geometry.
- The stations in a 2D Site Calibration shall not exceed a horizontal residual of 0.07 feet

3D Site Calibration:

In addition to the requirements described above for a 2D Site Calibration, the following requirements shall be met for a 3D Site Calibration:

- A minimum of 5 vertical (or 3D) control stations shall be included in a 3D Site Calibration. To avoid creation of a distorted or tipped plane, the stations selected must have been tied together with one common leveling circuit. An alternative to this requirement is to collect data on these 5 vertical control stations but include just one of them in the Site Calibration. Analysis of the data will determine which vertical control station represents a best-fit solution for the project. This may be a better alternative when working with vertical control that has not been recently tied together (OC Survey Benchmarks).
- Each vertical control station shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 4 hours.
- The stations in a 3D Site Calibration shall not exceed a vertical residual of 0.10 feet.

CHAPTER 9
HYDROGRAPHIC SURVEYS

General Statement

Hydrographic surveying is the science of measurement as it relates to the marine environment. A hydrographic survey may consist of one or more of the following: depth measurement, identification and charting of obstructions to navigation, determination of the nature of the bottom, measurement and characterization of the time and magnitude of current speed and direction, tidal measurements, determination of tidal datums, and positioning and charting of fixed objects. As the terminology related to hydrographic surveying is highly specialized, Appendix C provides a glossary with definitions of terms which appear within this document.

Purpose Statement

The purpose of this document is to provide general information and guidance on the conducting of hydrographic surveys by or for OC Survey. The specific focus will be on surveys which acquire general bathymetry, or those which measure or monitor dredging operations, and will be limited to projects with water depths of less than 600 feet. This document is not intended to supersede more authoritative manuals such as US Army Corps of Engineers Publication EM 1110-2-1003 or National Oceanic and Atmospheric Administration (NOAA) Field Procedures Manual.

Policy Statement

In lieu of establishing a rigid criteria for all hydrographic surveys, this document has been created for use as a "best practice" guide, in recognition of the fact that each project will have its own unique set of requirements and conditions, and no single accuracy standard, data collection procedure, or Quality Assurance/Quality Control (QA/QC) strategy can be defined which would cover all possible variables. Departure from the best practice recommendations laid out herein must be authorized by the Deputy County Surveyor, Field Services, or by the County Surveyor. The surveyor in responsible charge of a hydrographic survey performed by or for OC Survey should have a comprehensive working knowledge of this document, as well as the USACE and NOAA manuals referenced above, prior to conducting the survey.

Applications

Hydrographic surveying methods may be employed by or for OC Survey on the following types of projects:

- Scour Study Surveys
- Condition Surveys - County Lakes, Reservoirs, Harbors, and Navigation Channels
- Coastal Engineering Transect Surveys (beach profile surveys)
- Dredging Measurement and Monitoring Surveys (pre and post dredge)

Components of a Hydrographic Survey System

Hydrographic survey measurement systems can range from simple rod readings, lead line measurements positioned by a tagline, single beam echo soundings, to complex full coverage multibeam bathymetry survey systems. Modern hydrographic survey systems will include some combination of the equipment detailed below:

Survey Vessel:

The Survey vessel may range in size and complexity from small remotely operated systems up to large ocean-going vessels. A number of conditions must be considered when selecting the appropriate vessel for a specific project, including:

- Project site conditions – depth of water, current strength, wind strength, swell height, obstructions (such as dense piling or areas with limited overhead clearance), the need to portage over shallow areas, dense aquatic vegetation on or near the surface, density of vessel traffic in the area, etc.
- Project location in relation to the nearest launch facility
- Project budget
- Selected instrumentation to be employed for the survey

Acoustic Depth Measurement System:

Acoustic depth measurement systems compute depths by measuring the elapsed time an acoustic pulse takes to travel from the transducer to the seafloor and back. These measurements are converted to depth measurements by applying corrections to account for vessel static draft and dynamic draft, and for the sound speed at the survey location. The acoustic system should be tuned to best performance for specific project conditions by adjusting sound velocity, transmit power, pulse width, gain, and checked against redundant observations to validate proper tuning of the system. The following are the most commonly used types of acoustic depth measurement systems: (cont.)

Single Beam Systems – A single beam echosounder is a system capable of accurately measuring the water depth along the survey vessel track. The industry standard acoustic frequency for dredging and shallow water surveys is 200 kHz, which is the standard for determining the navigation depth and is approximately equivalent to a lead line sounding using an eight-pound mushroom anchor. When it is necessary to penetrate a suspended sediment layer or fluff associated with dredging operations, a dual frequency echosounder with a second transducer operating at 30 kHz is commonly used to determine navigational depths. System specifications should include a thermal printer or digital echogram, the ability to input index error and draft corrections, direct sound speed input, serial or Ethernet connectivity for rapid data transfer, and adjustments for transmit power, gain, and pulse width. Note: In order to meet industry standard of practice, in addition to collection of digital depths, all single beam systems should include a hard copy or digital echogram of the acoustic pulse. The echogram should be used during processing to verify that the digital depth algorithm has detected the actual seafloor, and not a second return from the bottom (multiple) or sonar returns from the water column (fish, bubbles, aquatic vegetation, suspended sediment, etc.). Commonly used single beam echosounder systems which meet these requirements include Teledyne ODOM CV100 (single frequency) and CV200 (dual frequency); CEE HydroSystems CEEPULSE (single frequency) and CEE ECHO (dual frequency); Knudsen Mini (single frequency) and 1602 (dual frequency). For more detailed information on single beam transducer systems, see **Chapter 4 - Single Beam Depth Measurement Systems** of the US Army Corps of Engineers Publication EM 1110-2-1003.

Multiple Transducer Systems - These systems consist of multiple single beam transducers, mounted perpendicular to the direction of travel, typically along a boom extending to either side of the vessel, to collect several single beam profiles along the vessel track. These systems are rarely used, typically applied in extremely shallow water were multibeam sonars are less effective, and require the use of a Motion Reference Unit (MRU) and a heading sensor (gyro). For more detailed information on multiple transducer systems see **Chapter 5 - Multiple Transducer Channel Sweep Systems for Shallow Draft Navigation Projects** of the US Army Corps of Engineers Publication EM 1110-2-1003.

Multibeam Systems - A multibeam sonar is an echosounder which transmits a wide acoustic pulse extending to either side of the vessel. The receive transducer array resolves the returning signal into numerous beams, recording two-way travel time and angle of arrival for each beam. In this way, accurate depth soundings are derived for the entire swath. The accuracy of a multibeam sonar is dependent upon application of corrections for vessel motion (e.g. pitch, roll, yaw, and heave) and spatial location, which is achieved by incorporation of high accuracy GNSS or inertial navigation systems. Consequently, multibeam sonars require real time integration with a number of other specialized sensor systems. Multibeam sonars are appropriate for use in a range of water depths and environments. Multibeam sonars can also provide backscatter information (e.g. reflected sound intensity or reflectance), which is useful for determining substrate characteristics. System specifications should include the ability to input draft corrections and sound speed (or the direct input from a sound speed instrument located at the sonar head, for flat array systems), Ethernet connectivity for rapid data transfer, the ability to apply precise timing from GNSS, and adjustments for transmit power, gain, and pulse width. Commonly used multibeam systems which meet these requirements include Teledyne Reson multibeam systems, R2Sonic multibeam systems, and Norbit multibeam systems. For more detailed information on multibeam systems see **Chapter 6 - Acoustic Multibeam Survey Systems** of the US Army Corps of Engineers Publication EM 1110-2-1003.

Sound Velocity Profiler:

Sound velocity is primarily influenced by water temperature and salinity and can vary over time and location due to tidal exchanges, mixing from tributaries, surface warming, upwelling, or other oceanographic and climatic influences. The capture of accurate acoustic depth measurements is dependent upon the use of a sound velocity profiler, which compensates for distortions in acoustic measurements attributed to these variances. A sound velocity profiler accurately measures the sound velocity profile through the water column, applies a sound velocity average to the instrumentation, and incorporates this profile into the processing software. On multibeam sonars, a sound velocity sensor is typically placed at the sonar head in order to accurately perform beam forming on flat array systems, in addition to taking sound velocity casts for a profile of sound velocity versus depth to correct for refraction and distance measurements (range). For single beam surveys a sound velocity profiler improves sounding accuracy over an average value and the full profile can be applied to correct depth measurements for variation in the sound speed profile. This is particularly important in estuaries where there may be a freshwater lens over denser salt water. Commonly used sound profiler systems include Teledyne ODOM Digibar, AML Oceanographic Smart SV&P, and Valeport MIDAS.

Motion Reference Unit (MRU) and Inertial Navigation System (INS):
Most hydrographic survey vessels are equipped with some type of motion compensation instrumentation, where wave induced motion is anticipated.
Motion Reference Units (MRUs) – MRUs measure and compensate for changes in heave, pitch, and roll. A gyro compass or dual antenna GNSS navigation system can be used with the MRU to provide a heading solution in the absence of an inertial navigation system (INS). Commonly used MRU systems which meet these requirements include: Teledyne TSS DMS-05 (heave and attitude) or Teledyne TSS MAHRS (heave, attitude and heading).

Inertial Navigation Systems (INS) – INS systems very accurately measure and compensate for heave, pitch, roll, and heading (yaw), and also integrate position and height data from GNSS into a coupled inertial solution, to provide reliable position and height data through short term GNSS outages. Commonly used INS systems which meet these requirements include Trimble Applanix POS/MV, iXblue Hydrins, and SBG Apogee-D.

For more detailed information on MRU and INS systems see C**hapter 7 - Section IV Vessel Motion and Orientation System** of the US Army Corps of Engineers Publication EM 1110-2-1003.

Positioning System:
The most widely accepted method for positioning of the survey vessel is Real Time Kinematic Global Navigation Satellite System (RTK GNSS). The survey vessel is outfitted with either one or two GNSS receivers, which are used to establish the horizontal and vertical position of the vessel, and to provide a heading solution in the case of the dual receiver array. GNSS systems have a time delay and should be integrated with the acquisition software by providing a precise time of observation message (e.g. NMEA, GZDA) in order to precisely time tag position messages. If satellite signal visibility is restricted or the survey area is subject to a high degree of multipath (e.g. bridges, overhead cranes, alongside ships, etc.), an INS system should be used to improve positioning accuracy. The alternative on smaller projects would be the integration of a robotic total station into the survey acquisition software solution. The total station should first be tested for time delays. An example of a robotic total station system that is readily integrated would be the Trimble SPS 930 with machine control capabilities. For more detailed information on positioning systems see **Chapter 7 - GPS Vessel Positioning, Orientation, and Water Surface Elevation Measurement** of the US Army Corps of Engineers Publication EM 1110-2-1003.

Data Acquisition and Navigation Display:

The computer system used for data acquisition and real-time navigation display should consist of a robust field PC or laptop with multiple network cards, serial ports, and USB ports, sufficient RAM and storage capacity, and a high-end video card. The PC or laptop must be capable of running industry standard hydrographic software (e.g. HYPACK®, QPS QINSy, Teledyne PDS) that precisely time tags data, provides real-time quality review of acquired data, and displays real-time navigation for conducting a systematic survey. A multibeam survey will require a more robust computer system than a single beam survey. Provided that space is available on the vessel, a rack mount PC with video monitors, keyboard and roll ball mouse is the preferred option. For vessels with limited space, a suitable specification laptop can be used for both multibeam and single beam surveys. Note that most modern laptops, with the exception of a few ruggedized models, are not equipped with standard external serial ports. In this instance, in order to reduce the potential for variable timing latency errors, it is recommended that a high quality USB-to-serial or Ethernet-to-serial adapter (e.g. Moxa or Quatech) be connected to the laptop to receive RS-232 serial messages from the survey sensors. The rack mount PC and laptop specifications suitable for single beam and multibeam bathymetry acquisition are tabulated below.

Recommended Rack Mount PC Specification for Single Beam and Multibeam Data Acquisition

Component	Single Beam	Multibeam
Processor	I7 Intel Quad Core	I7 Intel Quad Core
Memory	8 GB	32 GB
DVD Drive	Sata DVDRW	
Hard Drive	500 MB SSD	2 TB SSD
Graphics Card	2 GB memory & 2 display ports	8 GB memory & 4 display ports
Serial Ports	4 x COM ports 9-pin	4 x COM ports 9-pin
USB Ports	4 x USB 3.0 ports	4 x USB 3.0 ports
Network Ports	2 x gigabit Ethernet ports	2 x gigabit Ethernet ports
Operating System	Windows 10	Windows 10

Recommended Laptop Specification
for Single Beam and Multibeam Data Acquisition

Component	Single Beam	Multibeam
Processor	I7 Intel Quad Core	I7 Intel Quad Core
Memory	8 GB	32 GB
DVD Drive	Sata DVDRW	
Hard Drive	500 MB SSD	2 TB SSD
Graphics Card	2 GB memory & 1 display port	4 GB memory & 4 display ports
Serial Ports	4 port USB-to-serial or Ethernet-to-serial	4 port USB-to-serial or Ethernet-to-serial
USB Ports	2 x USB 3.0 ports	2 x USB 3.0 ports
Network Ports	1 x gigabit Ethernet port	1 x gigabit Ethernet port
Operating System	Windows 10	Windows 10

Selection of Depth Measurement System

Successful completion of a hydrographic survey project will depend upon selection of equipment appropriate to the required accuracy of the survey, redundant observations to document system accuracy and precision, and the impact of existing conditions. The discussion in this section will be limited to selection of the depth measurement system and the related components. Note that Multiple Transducer Systems are not included in the discussion as they have limited application.

Single Beam Systems:

Advantages:

- Inexpensive (an MRU is generally not required on protected and shallow water surveys)
- Correct application of RTK GNSS mitigates the impact of changes in static and dynamic draft, water level variability, and vessel heave
- Easy to operate
- Simple calibration procedure
- Can be operated in extremely shallow water (approximately 1 foot below the transducer)
- Accurate measurement, when properly calibrated and operated
- Effective in penetrating suspended sediment, which is inherent to dredging operations (with the use of a dual frequency 200/24 kHz echosounder)
- Processing of data is not complex

Disadvantages:

- Considered a cross-sectional survey, only generating a depth profile directly below the vessel track
- Longer data collection time to adequately map slopes and irregular bottom topography
- Ineffective at mapping obstructions
- Less effective at mapping steep slopes due to beam footprint and angle of incidence
- Less accurate representation of uneven seafloor topography
- Less accurate when vessel is subjected to heave, pitch and roll (in the absence of an MRU)
- Less accurate terrain model generated
- Inability to map under docks, vessels, booms, or other surface obstructions restricting vessel access
- May present possible pay quantity disputes when single beam bathymetry is compared to multibeam bathymetry

Best Uses:

- Coastal engineering transect surveys
- Channel condition surveys
- Dredging measurement surveys meeting the following conditions or requirements: aller project size, shallow water (d < 10 feet), uniform bottom topography, calm water conditions, lower accuracy requirements, and tight budget constraints
- Fast turnaround of interim dredge monitoring surveys
- QA tool, used as a performance test or independent check against a multibeam survey

Additional Notes:

- If a single beam system (200 kHz) is to be used without an MRU or properly applied RTK GNSS, surveys should only be conducted under ideal conditions, e.g. calm water (minimal tidal influence, swells, or wake from other vessels), minimal wind, etc.
- A high frequency transducer (200 kHz) will provide more precise depth measurements under optimal conditions and is the industry standard for navigation surveys; a low frequency transducer (30 kHz) is better able to penetrate aquatic vegetation and suspended sediment or fluid mud from dredging activity
- Transducers with a narrow beam width will provide higher resolution, especially when mapping slopes or irregular terrain, but will be more adversely affected by vessel pitch and roll (in the absence of an MRU); transducers with a wide beam width will create a larger footprint and thus are better suited for "strike detection", but tend to distort mapping of slopes and irregular terrain

- Single beam transmit power and receiver gain must be adjusted according to the ambient conditions, e.g. seafloor substrate, water column noise level, vessel noise, etc., in order to optimize the signal to noise ratio for accurate bottom detection

Multibeam Systems:

Advantages:

- Considered a full coverage survey (100% ensonification of the seafloor)
- Very fast rate of data collection
- More efficient than single beam surveys for channel mapping - longitudinal lines can be run, as opposed to cross sectional lines
- Accurate measurement of steep slopes and irregular seafloor
- Highly effective at mapping obstructions, provided methodology specifically applicable to object detection is followed
- Accurate terrain model generated

Disadvantages:

- Expensive – requires advanced sonar system and INS or MRU with additional heading sensor (gyro or dual GNSS)
- Complicated to operate - more crew training is required
- Complicated and time consuming calibration procedure; resultant data may be compromised if calibration is skipped or incorrectly conducted
- Frequent sound speed casts may be required to correct for sonar refraction
- Processing of data is complex and time consuming

Best Uses:

- Scour study surveys
- Wide area coastal sediment migration studies
- Channel condition surveys
- Dredging measurement surveys, especially those meeting the following conditions or requirements: larger project size, relatively deep water (d > 10 feet), object detection required for critical clearances, accurate mapping of steep slopes, mapping under obstructions (vessels, docks and piers), and high value (rock cut) dredging

Additional Considerations:

- Although multibeam transducers are capable of collecting wide sonar swaths, beam angles should be evaluated for effective swath width to meet required accuracies, as accuracy tends to degrade towards the outer portions of the swath. Effective swath is a function of the sonar system and the nature of the bottom, and can be dynamic based on angle of incidence of the sonar beam with the bottom (e.g. upslope beams can use a wider swath as they have a high angle of incidence while downslope beams would have a narrower swath as they have a low angle of incidence). A cross-line analysis should be conducted to determine effective swath width for the project. Use of a good quality sonar and integrated INS can result in a wider effective swath width.

Accuracy Standards

The accuracy standards recommended herein reflect those defined in **Chapter 3** of the US Army Corps of Engineers Publication EM 1110-2-1003. As stated in this USACE document, although the recommended procedural and accuracy standards reflect current equipment, positioning methodology, and QA/QC practices, "… it must be recognized that no single accuracy standard will be applicable to every (OC Survey) civil works project; therefore (OC Survey) should tailor their survey procedures and required accuracies to each specific project." Note that the **"National Oceanic and Atmospheric Administration (NOAA) Field Procedures Manual"** and the **"IHO Standards for Hydrographic Surveys"** were developed specifically for nautical charting purposes. Although they provide a valuable resource through the establishment of best practices and identification of uncertainties, they do not specifically apply to hydrographic surveys performed by OC Survey.

When project specifications state required accuracies for depth measurements, the statistical measurement criteria must also be defined, along with the calibration and compliance process to be followed. On dredging measurement surveys, project specifications shall require that performance tests and redundant observations be the basis for this confirmation (see Quality Control section below).

Recommended accuracy standards for OC Survey dredging measurement surveys up to a depth of 50 feet, with a soft bottom composition, are as follows:

- Repeatability of Depth Measurement: 0.3 feet
- Standard Deviation of Depth Measurement: +/- 0.8 feet
- Positioning System Horizontal Accuracy: 0.10 feet
- Positioning System Vertical Accuracy: 0.10 feet

Notes:

- These are recommended standards, which may be modified to meet specific project requirements or survey capabilities.
- Repeatability is computed from performance test data as the mean difference between cross-line check observations.
- Standard deviation of depth measurement is computed at the 95% confidence level from performance test data, e.g. cross-line check observations (single beam system) or coverage overlap (multibeam system).
- Horizontal and vertical accuracy of the positioning system are computed from the shoreline control point check shots (see Quality Assurance section below) and do not imply the same accuracy for the horizontal and vertical position of seafloor data points.
- Resultant accuracies should be reported within the project deliverables, along with a statement explaining the method used in computing the accuracies.

Elements of Hydrographic Survey Data

- Observations captured by any type of hydrographic survey system consist of the following basic elements:
- Horizontal position
- Elevation of the water surface or other reference point (vessel transducer, fixed dock or bridge, etc.)
- Depth measurement from the water surface or other reference point to the bottom

As all elements of an observation might not be conducted at the same precise moment in time, or might not be on the same time base (e.g. local time or Coordinated Universal Time - UTC), all observations should be recorded with a precise time stamp. Hydrographic acquisition and processing software is capable of correlating observations with time and applying documented time delays. This is particularly critical when observations are conducted from a moving vessel. The precise timing of instrumentation is best accomplished by synchronization with Global Navigation Satellite System (GNSS) time tags.

Sources of Uncertainty Unique to Depth Measurement

Uncertainty in depth measurements can be referred to collectively as Total Propagated Uncertainty (TPU). Much of this uncertainty can be modeled and thus mitigated by applying corrections to the measurement data. Specific depth measurement corrections are listed below, in the order which they are to be applied:

- Instrument error corrections - account for errors related to sounding equipment
- Static draft corrections - account for transducer depth below the water surface (when the vessel is at rest)
- Instrument spatial offsets (e.g. uncertainty in measurements of offsets from phase center of the GNSS antenna to transducer acoustic measurement reference point)
- Dynamic draft corrections - account for vertical displacement of the transducer when the vessel is underway
- Speed of sound corrections - account for variability in the sound speed profile
- Attitude corrections - account for vessel and transducer motion, and heading bias relative to measured heading,
- Timing (latency) corrections – account for latency of positioning data

Use of other calibration measures, e.g. the “bar check” or “ball check” will quantify the transducer index constant (see **Quality Control** section below). Resultant accuracy, and thus repeatability, can then be computed after performance testing has been completed (see Quality Assurance section below).

Other sources of uncertainty which can be minimized through beam width, frequency, transmit power, and gain settings include:

- Varying magnitudes of noise in the acoustic signal
- Variations in the amount of suspended sediment within the water column
- Variations in seafloor material (reflectivity or aquatic vegetation)
- Irregularity of seafloor topography

Quality Control Procedures (QC)

The QC procedures detailed below will help to improve the accuracy of depth measurements by removing systematic uncertainties and random motion related positioning errors.

Calibration Specific to Single Beam Systems:

Dock Check:

The Dock Check serves as the initial check on the entire hydrographic system. This check is performed by comparing two measurements to a common point on the seafloor adjacent to the dock, one using a survey layout rod and one using the hydrographic system itself. The dock check shall be performed at the beginning of each day, throughout the duration of the project.

Calibration Specific to Multibeam Systems:

Patch Test:

A patch test shall be conducted for each survey vessel to confirm alignment of the IMU sensor with the sonar transducer and to verify delay times applied to the time-tagged sensor data. The patch test consists of a series of lines run in a specific pattern, then used in pairs to analyze roll, pitch, and heading alignment bias angles, as well as latency (time delays) in the time tagging of the sensor data. The patch test lines are to be run in accordance with USACE specifications and evaluated in the following order: latency, roll, pitch, and heading. A precise timing latency test is performed by running lines in the same direction, at different speeds, over a steep slope or other prominent topographic feature. Roll alignment is determined by running reciprocal lines over a flat bottom, in the deepest part of the survey area. Pitch bias for multibeam systems is determined by running reciprocal lines over a smooth slope or noticeable feature, perpendicular to the depth curves. Heading bias is determined by running two or more adjacent pairs of reciprocal lines on each side of a submerged object or feature, in relatively shallow water. Lines are to be run at a speed which allows for forward overlap. Once final mounting angles have been determined, the values are entered into the hydrographic acquisition software and verified during processing. A confidence check shall be performed to verify that accuracy requirements have been met. Patch tests shall be repeated whenever changes are made to equipment hardware or software, sensor failure, replacement, or whenever assessment of the data indicates that system accuracies do not meet specified requirements.

Calibration Applicable to Both Single Beam and Multibeam Systems:

Alignment and Calibration of the Motion Reference Unit (MRU):

Upon installation, components of the MRU must be properly aligned and calibrated. A thorough explanation of this process can be found in Chapter 6 of US Army Corps of Engineers Publication EM 1110-2-1003.

Vessel Baseline Survey:

A vessel baseline survey shall be performed prior to survey operations by surveying sensor offsets relative to the vessel reference point and checked by redundant observations (e.g. independent total station observations from a different setup, or measurements of vertical offset, in meters and feet, if the GNSS antenna is mounted directly over the transducer). The purpose of these redundant measurements is to confirm that the measurements were made correctly and to determine the uncertainty associated with the measurements. The sensor offset values calculated during the baseline survey shall be used for all surveys performed for the project. Measurement offsets derived from the baseline survey should be entered into the hydrographic acquisition software and verified during processing (Note: It is recommended that offset values not be entered directly into the GNSS or single beam echosounder, as errors entered there will be more difficult to rectify than those entered into the hydrographic acquisition software). If the GNSS system is mounted directly over the single beam transducer, a direct measurement is made from the antenna phase center to the bottom face of the transducer. This distance shall be measured in feet and in meters, with the metric-to-imperial conversion serving as a redundant check.

Bar Check and Draft Measurement Comparison (Static Draft):

This comparison is used to minimize systematic errors introduced by instrumentation and draft measurements. While secured to the dock, or in protected water with little or no current and adequate water depth, draft relative to the transducer (multibeam or single beam) shall be recorded from port and starboard draft marks abeam of the transducer mount. A sound speed profile should be collected prior to the bar check and applied in the hydrographic software package (preferred method) or the average profile value applied in the echosounder hardware (less preferred method) to correct the resultant depths for changes in water column speed of sound. A bar is lowered to a suitable depth below the echosounder transducer, and a physically measured depth is compared to the instrument reported depth. The resultant difference between these measurements is entered into the hydrographic acquisition software, entered directly into the echosounder (less preferred method), or recorded for post-processing. In addition, it is imperative that a log book documenting results of the bar check be maintained. An alternative to the bar check, especially well-suited to an over-the-side mounted transducer, is the Ball Check. The “ball” (or a flat plate) is suspended beneath the transducer as described above. The bar/ball Check shall be performed upon assembly of the hydrographic survey system, at the beginning of each project, and at regular intervals throughout the project (to be determined by the Senior Land Surveyor).

Vessel Settlement and Squat Test (Dynamic Draft):
A settlement and squat test shall be performed for the survey vessel prior to survey operations using post-processed kinematic (PPK) GNSS techniques. Dynamic draft measurements shall be measured to a precision of +/- 1 centimeter. Settlement and squat values are used as a quality measure to compare to other water level observations and are not to be applied when project elevations will be derived by RTK GNSS.

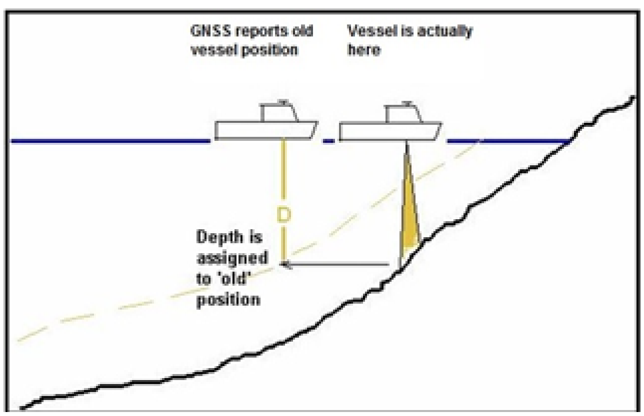
Sound Speed Profiles:
A sound speed profile system measures the sound speed profile in the water column, applies a sound speed to the instrumentation to correct the resultant depths for water sound speed variations, and incorporates this profile into the processing software. Sound speed profile observations shall be performed at least twice a day throughout the project, or more often if variations in the sound speed profiles are observed. Full depth sound speed profiles should be collected within the survey limits, within adjacent calibration sites (i.e. bar check and dock check sites), and within any areas of suspected change. Full depth profiles must also be applied if there are significant changes in the profile due to thermoclines or haloclines, or significant spatial or temporal changes in the sound speed profile relative to the average sound speed profile. If applicable, a real-time comparison shall be made between the sound speed profiler near surface sound speed to the sound speed measured at the sonar head, with any deviation noted in the log. It is imperative that a log book documenting results of sound speed profile tests be maintained. The sound speed profile system should be calibrated annually by the sensor manufacturer or approved company and a weekly comparison of the sound speed and depth against another sensor or by measure the sound speed in distilled water, measuring the temperature with a calibrated digital thermometer, and use a table for speed of sound vs. temperature.

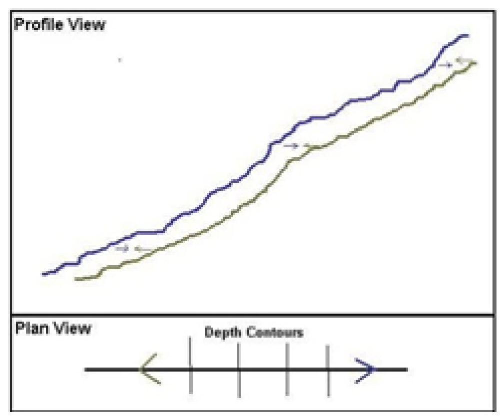
Latency Test:
The latency test is performed to identify time related bias due to delay between the GNSS time tag and the echosounder time tag. The latency test shall be performed upon assembly of the hydrographic survey system, at the beginning of each project, and in the event instrumentation is changed during a survey. The latency test involves surveying reciprocal lines at a constant speed up or down a slope or over a prominent topographic feature. Note that the latency test line direction for single beam systems is different from that for multibeam systems in order to differentiate pitch bias from latency. Presence of a latency error will result in these sounding profiles being offset from one another. Time offsets may also be present if the vessel experiences significant pitch when running at higher speeds.

The latency test should be conducted at a low rate of speed at slack tide when the water level is changing slowly, and only under optimal GNSS conditions (low DOP, high number of visible satellites, etc.) The planned survey lines should be followed as closely as possible. Ideally the latency test should be run three times, each with a different pair of lines, and the average latency value calculated from these independent tests. A thorough explanation of this process can be found in **Chapter 2** of the HYPACK® User Manual.

If using a pole mounted transducer and the RTK GNSS antenna is mounted directly above the transducer on a removable sonar pole, a more accurate latency test may be conducted by moving the pole horizontally and vertically one to two feet in each direction at an interval of one cycle every one to two seconds. If RTK GNSS height data is precisely time tagged with the soundings (zero latency) the resultant data will resemble a level plane. If this test results in an uneven surface, the latency correction is adjusted and the test repeated until a level plane is achieved. This test is best performed from a dock.

Effects of latency errors on the sounding profile





Quality Assurance Procedures (QA)

Quality Assurance (QA) is achieved through performance testing, both inside and outside of the project area. Performance tests capture redundant measurements, allowing for computation and documentation of repeatability and reliability of depth measurements. Results of performance tests provide verification that project accuracy standards have been met, and facilitate documentation of the actual reported accuracy of the survey. In order to compute a realistic statistical analysis, at least 100 pairs of points should be analyzed. Note that results of statistical analysis on non-independent data (data captured by the same measurement system) are only an estimate of system precision and do not truly represent a model of the accuracy of the data. Non-independent tests will not reveal constant system bias (systematic errors). Documentation of the results of the performance tests detailed below shall be included in the survey report.

Position Confidence Checks:

Horizontal and vertical accuracy of the positioning system itself are computed from check shots taken on shoreline control points using a survey layout rod. These check shots shall be performed at the beginning and end of each day, throughout the duration of the project.

Independent Depth Observations:

With the vessel secured to the dock, RTK GNSS corrected multibeam or single beam data is logged. A lead line or pole sounding is acquired at the multibeam sonar head or single beam transducer and an RTK GNSS water line observation is simultaneously acquired. After applying the manual sounding depth to the measured water level, the resultant bottom elevation is compared to the multibeam or single beam elevation directly below the sonar. The independent depth observation shall be performed at the start of each project and in the event instrumentation is changed during the project.

Crossline Comparisons (single beam and multibeam systems):

Crossline comparisons shall be conducted to ensure that sensor biases, GNSS height data, and sound speed profiles are accounted for in the data set. A statistical analysis of the crossline comparisons shall be performed, compiling statistics by beam number for multibeam systems and by crossing for single beam systems. In addition, a statistical analysis of the cross-line data to main scheme survey lines shall be performed and included in the Survey Report.

Comparison to Prior Survey (single beam and multibeam systems):

This comparison is accomplished by extending the pre and post dredge surveys beyond the anticipated dredge influence area. For condition surveys, current project data is compared with data from a prior survey, utilizing areas which would be resistant to change for the comparison (e.g. bridge footings, rock outcrops, etc.). A statistical analysis of the data within these areas shall be performed and included in the Survey Report.

Data Overlap (multibeam systems):

Survey lines run with a multibeam system are designed to provide a pre-determined percentage of data overlap. The data falling within these overlap areas provides the basis for performance testing. An independent performance test can be conducted by collecting single beam data within the multibeam data set. A statistical analysis of the data within these areas shall be performed and included in the Survey Report.

Sound Speed Profile Confidence Checks (single beam and multibeam surveys): Weekly confidence checks shall be performed by comparing depth and sound speed data from the primary sound speed profiler with those made from a secondary system at the start of the project. An alternative would be a comparison to sound speed measurements made in distilled water with the temperature measured with a precision digital thermometer and compared to a standard sound speed table. Deviations in these checks shall not exceed 2 meters/second (6 feet per second).

Survey Control Network

Horizontal and vertical positioning of the vessel is typically established by Real Time Kinematic (RTK) or Post Processed Kinematic (PPK) GNSS in conformance with Chapter 2 – RTK GNSS. The RTK/PPK GNSS survey should be tied to a survey control network which is based upon the California Coordinate System of 1983 (CCS83); however some projects may require a tie to a different coordinate basis, such as NAD27. Following are details related to establishment of the survey control network.

Horizontal Control:

The RTK GNSS survey may be based upon the the California Real Time Network (CRTN) or a traditional base/rover configuration.

In the event that CRTN is used and it is necessary to constrain to existing project control or to a coordinate system or epoch not supported by the RTN, a site calibration shall be performed (see Appendix A). Note that if a site calibration is utilized and project data is to be collected using hydrographic data acquisition and processing software (e.g. HYPACK®), coordinate system parameters established by the site calibration shall be entered into the hydrographic software and not applied directly to raw latitude/longitude values.

If the decision is made to use a base/rover configuration and there is no existing project control, the base station and other shoreline control points within the project area shall be established by static GNSS in conformance with Chapter 1 – Static GNSS. If existing project control is to be used as the basis of the horizontal control scheme, the integrity of the network shall be verified before the hydrographic survey commences.

Vertical Control:

Elevations of Temporary Benchmarks (TBMs) and shoreline control points shall be established by OC Survey Third Order Leveling procedures in conformance with Chapter 4 – Differential Leveling. If existing project control is to be used as the basis of the vertical control scheme, the integrity of the network shall be verified before the hydrographic survey commences.

Vertical Datum:
Hydrographic surveys are often tied to a different vertical datum (e.g. Mean Lower Low Water) than that of conventional surveys performed by OC Survey. It is essential that elevations of project control points are converted to the desired vertical datum. Appendix B provides a graphical depiction of the relationship between commonly used vertical datums. In the event that CRTN is used and it is necessary to constrain to a vertical datum not supported by the RTN, a single-point vertical site calibration shall be performed (see Appendix A). Note that if a vertical site calibration is utilized and project data is to be collected using hydrographic data acquisition and processing software (e.g. HYPACK®), the vertical correction established by the site calibration shall be entered into the hydrographic software as a static offset and not applied directly to raw ellipsoid height values. Possible alternatives to vertical site calibration are utilization of NOAA's Vertical Datum Transformation software (VDatum) or creation of a user-defined transformation grid (e.g. HYPACK® KTD grid).

Geoid Model:
Surveys which encompass a large geographic area shall incorporate a geoid model (e.g. Geoid12b).

Monumentation

Monuments set as control points during the course of a hydrographic survey shall meet the following criteria:

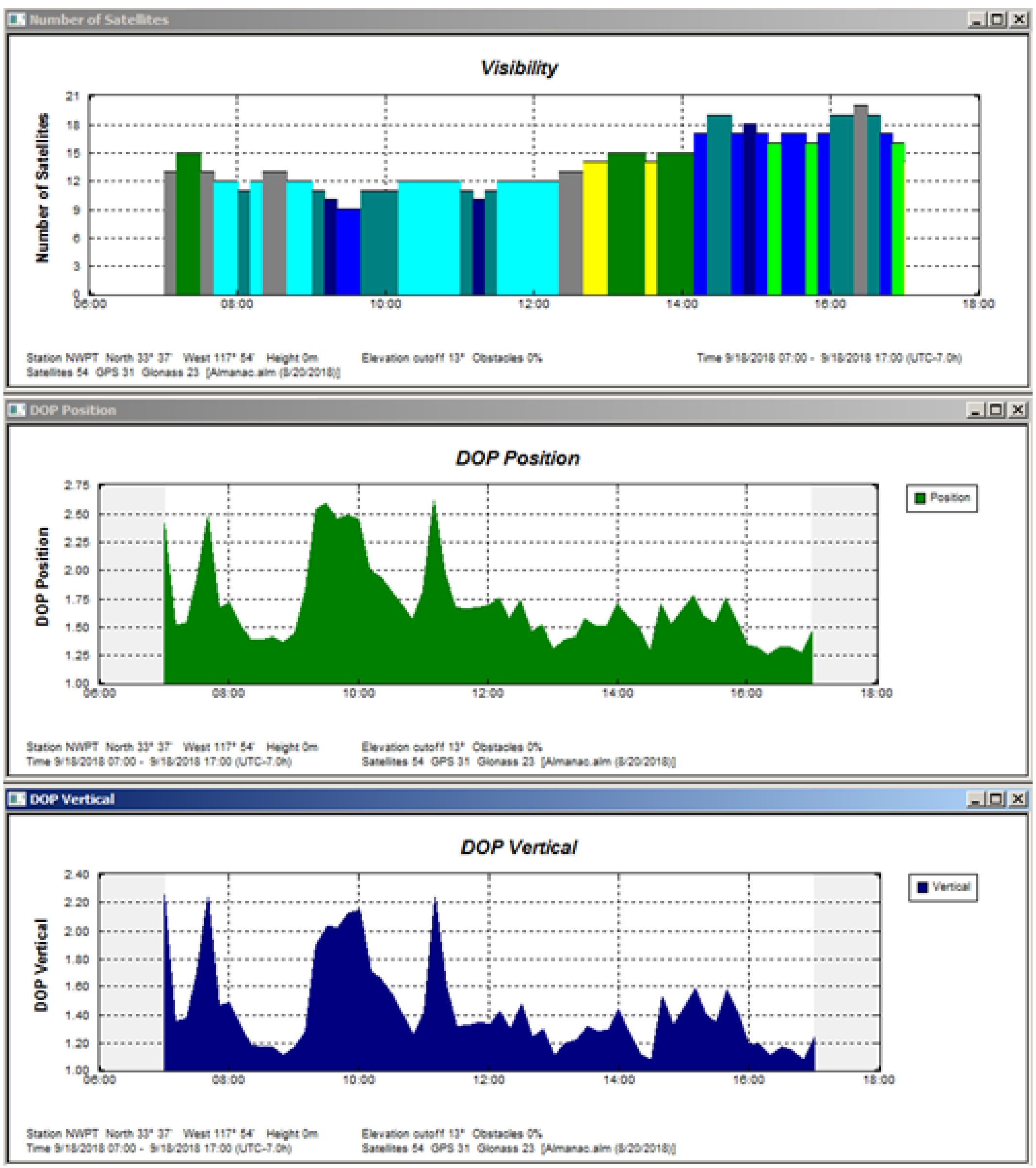
- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of an iron pipe with a tag or disk, or a rebar with an aluminum cap. Rebar must be set a minimum of 3 inches below the ground surface.
- All tags/washers/disks/caps referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge, and shall also be stamped "CP" or "CONTROL POINT".
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe or rebar.

RTK Survey Requirements for Positioning of the Vessel

Below are specific requirements related to positioning of the vessel:

- For each day of the survey, planning software shall be utilized to identify and avoid periods of high DOP, especially in the vertical component (see Figure 1 below)
- Whenever possible, the RTN base station selected should include all available satellite systems (GPS + GLONASS + etc.)
- Whenever possible, a PPK survey should be conducted in lieu of a conventional RTK survey, provided the processing software is capable of integrating PPK data
- Elevation mask shall be set to 15 degrees
- The GNSS base station selected should be no more than 10 miles from the project location

Figure 1 – Planning the RTK Survey



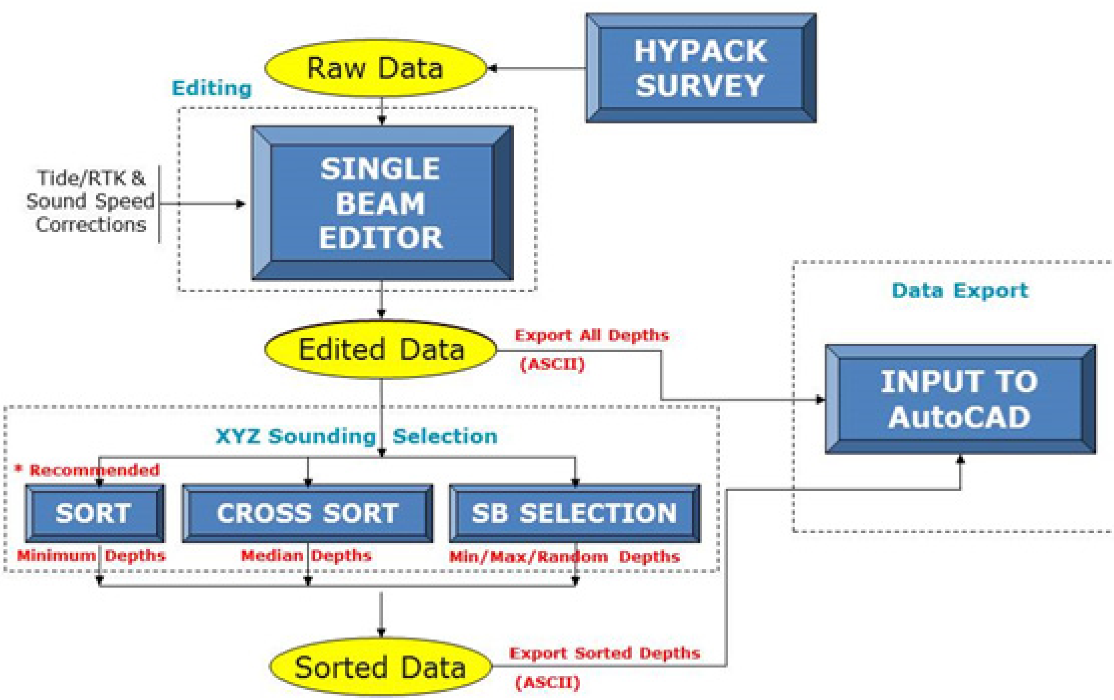
Data Processing

Although data processing styles will vary depending upon the software platform used, some basics are covered below.

Single Beam Data Processing:

Single beam bathymetry should be collected and processed using HYPACK® software. HYPACK® facilitates the processing of raw bathymetry into final hydrographic data products (sorted XYZ data, contours, cross sections, volumes, and CADD output) using various workflow routines and data editing processes. Refer to Figure 2 below for a simplified single beam processing workflow. A thorough explanation of this process can be found in **Chapter 4** of the HYPACK® User Manual.

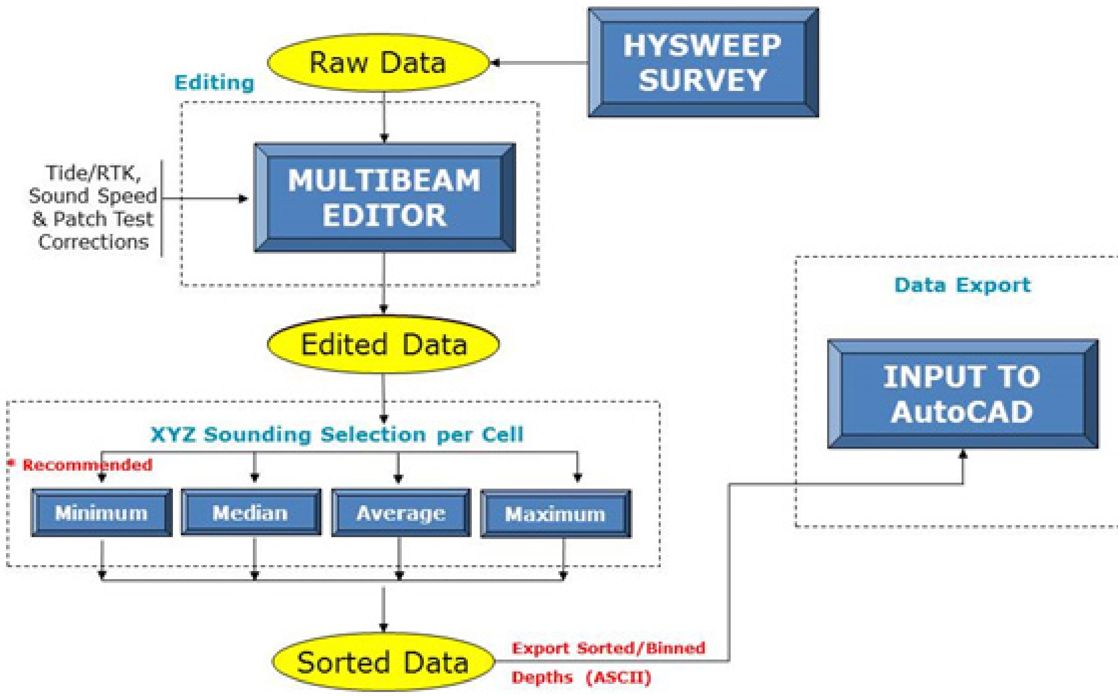
Figure 2 - Single Beam Bathymetry Processing Workflow in HYPACK® software



Multibeam Data Processing:

Multibeam bathymetry should be collected and processed using HYPACK® software. HYPACK® facilitates the processing of raw bathymetry into final hydrographic data products (sorted XYZ data, contours, cross sections, volumes, and CADD output) using various workflow routines and data editing processes. Refer to Figure 3 below for a simplified multibeam processing workflow. A thorough explanation of this process can be found in **Chapter 6** of the HYPACK® User Manual.

Figure 3 - Multibeam Bathymetry Processing Workflow in HYPACK® software



Deliverables

Expected deliverables will vary from project to project, thus it is imperative that the Project Management team unambiguously defines the delivery and accuracy requirements at the project planning phase. OC Survey staff must have a thorough understanding of and be in complete agreement with these requirements before commencement of the project.

TIN Model:

The primary deliverable from a hydrographic survey project is a Triangulated Irregular Network (TIN) model. This is a 3D representation of the existing sea floor conditions at the time of the survey. Dredging measurement surveys will be composed of two independent surveys – a pre- dredge survey and a post-dredge survey. After delivery of the pre-dredge TIN model, the Project Resident Engineer (RE) will define the payment prism, over-depth payment prism, side slope allowance, and box cut allowance. These definitions will be used in conjunction with the post- dredge TIN model to compute the volume of material removed which is eligible for payment. Volume computations are based upon a surface to surface comparison between the pre-dredge and post dredge surfaces.

Survey Report:

The Survey Report is a detailed compilation of project details, including but not limited to the following:

- Project overview / statement of purpose
- Components of the hydrographic system used for the survey
- Survey control diagrams and a narrative of the horizontal and control scheme employed
- Detailed description of the QC procedures employed, including sound speed profiles, calibration of equipment and results of the bar check, latency test, etc.
- Detailed description of the QA procedures employed, including position confidence checks, independent depth observations, crossline comparisons, etc.
- Log book entries and other field notes

Other deliverables:

Other deliverables may include sounding charts, color-banded depictions of contours, cross sections, etc.

Additional Resources

- US Army Corps of Engineers Publication EM 1110-2-1003
- National Oceanic and Atmospheric Administration (NOAA) Field Procedures Manual
- International Hydrographic Organization (IHO) Standards for Hydrographic Surveys
- HYPACK® User Manual

Appendix A - Site Calibration Procedure

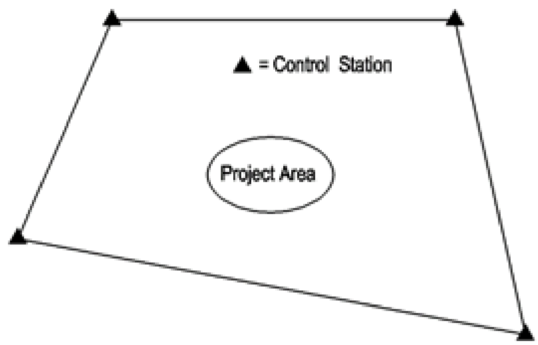
A Site Calibration establishes a relationship between the observed WGS84 coordinates and the local grid coordinates.

General Requirements:

- The control stations shall be selected so as to create a polygon which fully encompasses the project area (see Figure 4). Selected control stations must be of the same epoch date as the current project and be located no more than six miles from the CRTN Station or Base Station.
- Conditions which may generate multipath or obstruct view of the satellites, such as overhead power lines, nearby trees, or adjacent buildings, should be avoided.
- Elevation mask shall be set to 15 degrees.
- Each occupation shall consist of either one measurement of 180 epochs, or three sequential measurements of 60 epochs each.

- Upon computation of the Site Calibration, a control station with residual values greater than those defined below shall be discarded and another control station shall be used in place of this outlier.
- All subsequent measurements and staking activities shall use the same CRTN Base Station or Base position as was used to generate the Site Calibration.

Figure 4 – Control Point Selection



2D Site Calibration:

- A minimum of 4 horizontal control stations shall be included in a 2D Site Calibration.
- Each horizontal control station shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 2 hours. These time differentials are required in order to ensure significantly different satellite geometry.
- The stations in a 2D Site Calibration shall not exceed a horizontal residual of 0.07 feet

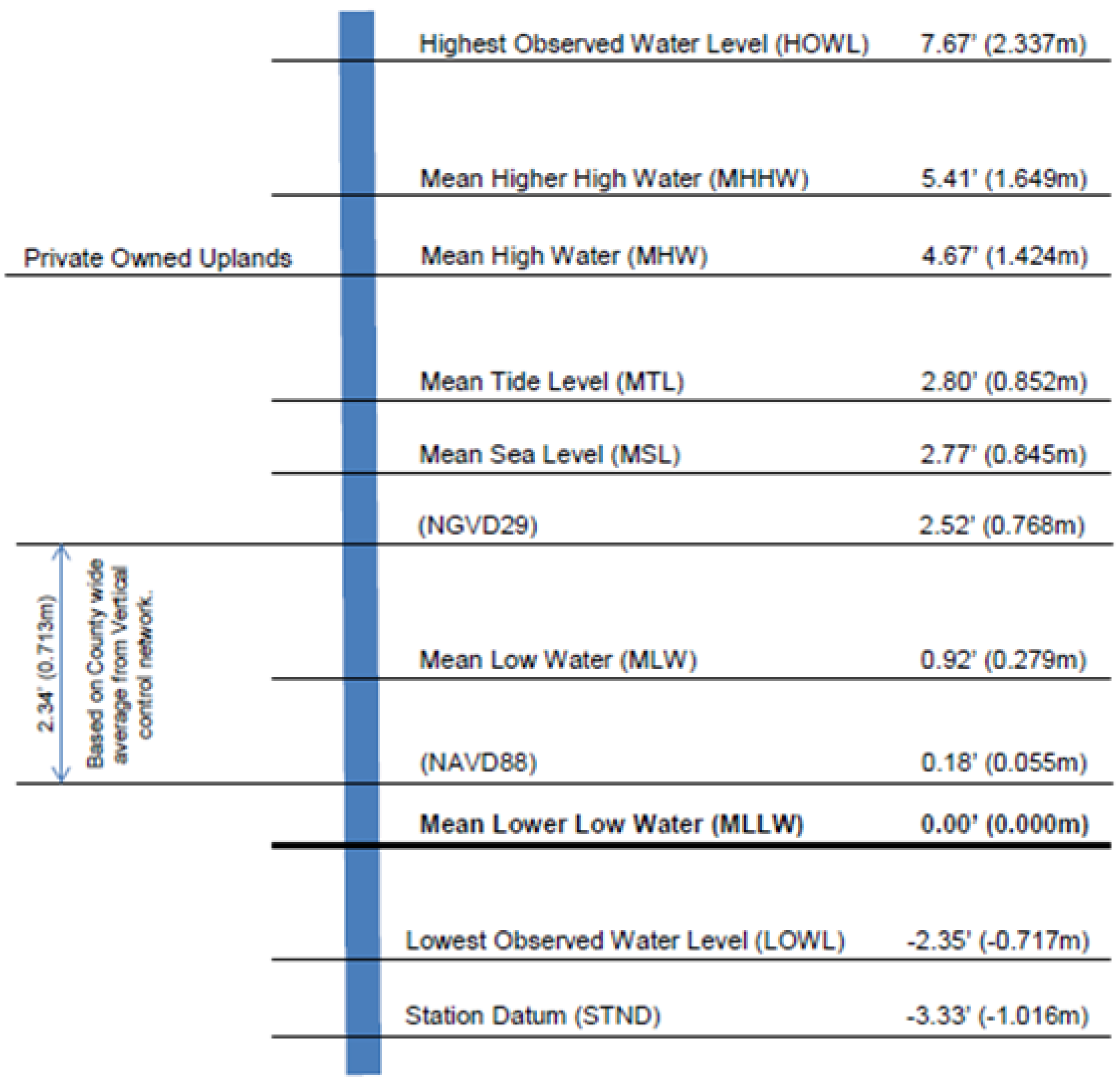
3D Site Calibration:

- In addition to the requirements described above for a 2D Site Calibration, the following requirements shall be met for a 3D Site Calibration:
- A minimum of 5 vertical (or 3D) control stations shall be included in a 3D Site Calibration. To avoid creation of a distorted or tipped plane, the stations selected must have been tied together with one common leveling circuit. An alternative to this requirement is to collect data on these 5 vertical control stations but include just one of them in the Site Calibration. Analysis of the data will determine which vertical control station represents a best-fit solution for the project. This may be a better alternative when working with vertical control that has not been recently tied together (OC Survey Benchmarks).
- Each vertical control station shall be measured with 2 independent occupations, with a minimum time differential (time of day) of 4 hours.
- The stations in a 3D Site Calibration shall not exceed a vertical residual of 0.10 feet.

Appendix B – Tidal Datum Relationships

The chart below displays tidal datums (in US Survey Feet) for benchmark "TIDAL-1NP", which is located at Newport Bay entrance, and is based on the following:

Station ID	9410580
Status	Accepted (Apr 17 2003)
Length of Series	14 years
Time Period	1980-1994
Tidal Epoch	1983-2001
Datum	STND



Information was taken from National Ocean Service (NOS), Tides and Currents website October 29, 2015

Appendix C - Glossary

Angle of Incidence:
The angle that a straight line meeting a surface makes with a line which is normal to that surface

Attitude:
The orientation of a vessel with respect to its longitudinal, transverse, and vertical axes

Backscatter:
The reflection of waves, particles, or signals back in the direction from which they came

Bathymetry:
The measurement of depths of water; also the information derived from such measurements

Beam Forming:
The process of shaping an acoustic beam through the control of the geometry of the transducer array

Bias:
The distortion of a result due to the introduction of a systematic error

Coordinated Universal Time (UTC):
A time reference based upon atomic clocks which is approximately equal to Greenwich Mean Time

Crosslines:
Sounding lines which cross the main system of survey lines

Depth Curves:
Lines on a map or nautical chart which generally connect points of equal depth at or below a specified datum

Dilution of Precision (DOP):
A measure of the effect of satellite geometry on the precision of positional measurements

Draft (Static Draft):
The vertical distance from the surface of the water to the transducer face
Dynamic Draft:
Draft of a vessel when underway

Echogram:
A graphic representation of the sea floor profile as recorded by the echosounder

Echosounder:
An instrument which computes water depths by measuring the elapsed time an acoustic pulse takes to travel from the transducer to the sea floor and back

Ensonification:
Exposure of an area of seafloor to acoustic energy

Gain:
A measure of signal amplification

Geoid:
An equipotential surface which best represents mean sea level

Geoid Model:
A mathematical representation of the geoid which is used to compute orthometric heights (elevations) from GNSS derived ellipsoidal heights

Halocline:
A layer in the water column where the vertical salinity gradient is steeper than that above it or below it

Heading:
The direction in which the longitudinal axis of a vessel is pointing at a given instant in time

Heave:
The rise and fall of a vessel due to the lifting force of the sea

Lead Line:
A graduated line attached to a lead weight which is used to determine the depth of water

Mainscheme:
The primary set of sounding lines in a hydrographic survey

Noise:
Extraneous signals detected by a sonar system

Pitch:
The alternate rise and fall of the bow and stern of a vessel about its transverse axis

Refraction:
The change in direction of a sonar pulse due to discontinuity of temperature and/or salinity in the water column

Roll:
The tilting rotation of a vessel about its longitudinal axis

Settlement:
Vertical displacement of a moving vessel, relative to its level when motionless

Sounding(s):
The act of measuring depth of water; also the information derived from such an action

Squat:
A hydrodynamic phenomenon by which a vessel moving through shallow water creates an area of lowered pressure that causes the vessel to be closer to the seafloor than expected

Swath Width:
The lateral coverage of a sonar system on the seafloor

Tagline:
A line used to facilitate the taking of equally spaced soundings

Thermocline:
A layer in the water column where the vertical temperature gradient is steeper than that above it or below it

Transducer:
The component of a sonar system which converts electrical energy to acoustic energy

Water Column:
A vertical section of water from the surface to the seafloor

Yaw:
The rotation of a vessel about its vertical axis, affecting heading

CHAPTER 10
SMALL UNMANNED AERIAL VEHICLE (sUAV) SURVEYS

Policy Statement

All projects which involve flights of small Unmanned Aerial Vehicles (sUAV, also known as "drones") will be performed in conformance with the specifications as defined in this document.

General Statement

sUAV may be employed for engineering-related surveys, including topographic surveys (limited use), volumetric surveys, and scour study surveys, provided the policy outlined herein is followed. In addition to the surveys listed above, sUAV may be used for project documentation, monitoring, inspection, and emergency response. The use of sUAV presents a new and rapidly evolving technology and therefore this document will need to evolve as well.

Acronyms/Definitions

AGL – Above Ground Level: Refers to flying height above ground

Airspace – Regulated by the FAA: there are six Classes of airspace (A, B, C, D, E, & G) and four Types of airspace (Controlled, Uncontrolled, Special Use, & Other). Class G is Uncontrolled and can be flown at any time during the day under 400 feet AGL, as long as it is safe to do so. All other classes are Controlled and may require FAA authorization for flight.

FAA - Federal Aviation Administration

LAANC - Low Altitude Authorization and Notification Capability: A collaboration between FAA and private industry which directly supports integration of UAS into the airspace.

PIC - Pilot in Command: The individual who has the final authority and responsibility for the entire operation and safety (the PIC must have a valid RPC)

Pilot - Person flying and controlling the sUAV (this does not require an RPC)

RPC - Remote Pilot Certificate: An FAA airman certificate specifically for the operation of sUAV

sUAV - Small Unmanned Aerial Vehicle

sUAV Team - Flight crew consisting of at least two people, but preferably three: the PIC, the Pilot, and the VR (the PIC must have a valid RPC)

UAS - Unmanned Aircraft System

VO - Visual Observer: Person who maintains constant line of sight with sUAV and communicates with the Pilot regarding airspace conditions, obstructions, and other aerial vehicles (this does not require an RPC)

Flight Plan

- Before a flight can be conducted, the sUAV Team shall determine in which airspace the flight will occur and identify any FAA restrictions which affect the flight. If necessary, FAA authorization shall be requested through LAANC via an automated application/approval process developed by an FAA-Approved UAS Service Supplier (USS). If approved, authorization is received in near-real time.
- The sUAV Team will ensure that the sUAV is in proper working condition and that all batteries are charged.
- The sUAV Team shall carry the "DRONE PROGRAM" 3-ring binder at all times. This binder contains necessary paperwork, such as sUAV Preflight Check List, RPC certificates, sUAV Registrations, and Repair Log.

Pre-Flight

- When arriving at the flight location, the sUAV Team will work together to inspect and discuss the airspace and note any weather issues or obstructions such as trees, buildings, power lines, etc.
- The PIC will notify the person(s) in charge of the project (project manager, construction superintendent, etc.), if present, that a flight will occur and discuss any possible issues or concerns.
- The PIC will determine whether or not it is safe to fly.
- Before flight, the sUAV Team will review and follow all "sUAV Preflight Check List" procedures (see Appendix A).
- During flight, the VR will closely observe the sUAV and communicate with the Pilot regarding current conditions and concerns. Any aircraft in the area will be discussed.

Post-Flight

- The sUAV Team will inspect the sUAV for any issues and note which batteries need to be charged.
- All Flight Log information pertaining to the flight such as Project, Date/Time, Flight Duration, Conditions, Drone, Pilots, etc., will be documented by using an ESRI mobile application on the flight IPAD. The application currently in use is named "***Survey123***" can be downloaded from the App Store. Once downloaded, the user logs into the app with their ESRI OCPW account and downloads the survey named "***OC Survey sUAV Flight Log***". The user then fills in and answers all questions pertaining to the flight and then submits/uploads the form to the ESRI server. This information can then be viewed back in the office on their PC.

Policy for Engineering-Related Surveys

sUAV may be used for engineering-related surveys, provided that policy detailed below is followed:

General Policy

- Cameras mounted on sUAV are not capable of consistently or reliably penetrating water. Topographic features which are submerged must be captured conventionally and merged with the data collected by the sUAV.
- Cameras mounted on sUAV are not capable of penetrating foliage. Topographic features which are obscured by foliage must be captured conventionally and merged with the data collected by the sUAV. Surface models may only be generated after trees, shrubbery, cars, etc. are isolated on separate layers or regions.
- Cameras mounted on sUAV are not capable of accurately identifying breaklines, such as top of curb, flowline, retaining walls, etc., and adjacent features which lack significant color contrast cannot be reliably identified and segregated.
- Specific policy details outlined below are based upon rigorous testing with camera-equipped sUAV only. Use of LiDAR-equipped sUAV will be the subject of future study, at which time an update to this document will be released.
- For information on the testing program employed by OC Survey in the development of this policy, refer to the following documents:
  - sUAV Testing (Camera) - Equipment and Procedures – details specific to the execution and processing of test flights
  - sUAV Testing (Camera) - Data Compilation – a compilation of statistical analysis of test data

Horizontal and Vertical Control - Establishment of Ground Control Point (GCP) Positions

- For projects which rely solely on sUAV for data, horizontal and vertical components of GCP positions may be established by GNSS (RTK or static), conventional traverse, or a combination of these methods. GNSS derived elevations shall be established using an NGS published geoid model.
- For projects which combine sUAV data with data of a higher order (e.g. hard surface topo collected with a total station and merged with supplemental sUAV data), project control shall be established as directed in Chapter 6 – Topographic Surveys, and GCPs shall be based upon this project control.

Ground Control Point (GCP) Placement

- All projects shall include at least five GCPs. Large projects or linear corridors shall include additional GCPs as needed to result in a maximum spacing of 300 feet.
- When terrain has significant vertical relief such as hills and/or canyons, GCPs shall be set near changes in vertical relief, such as bottom and top of hill, bottom and top of canyon, etc.
- Projects which include multiple surface planes (e.g. stockpile or earthwork removal) shall have a minimum of three GCPs on each surface plane.
- A minimum of three additional (targeted or photo ID) QC check-points shall be included for all projects.
- GCP targets may consist of any material, provided the target itself creates a clearly discernable contrasting color and a clearly discernable center point (e.g. an eight-inch diameter red plastic disk with a nail and tin at the center).

Collection and Processing of Data

- Generally speaking, the lower the flying height, the higher the resolution and thus the higher the resultant accuracy. Flying height shall be set as low as possible so as to maintain an acceptable balance between accuracy and flight time. The optimal flying height would be between 80 and 120 feet above AGL. The flying height may be increased when accuracy requirements are less stringent.
- PPK data shall be collected for all projects, provided the sUAV is equipped with a PPK-capable GNSS receiver. PPK data shall be logged at 10 Hz (0.1 seconds); CGPS base stations shall log at 1 Hz (1 second).
- Data may be processed and aligned using various control combinations, depending on the project. These combinations include: (1) constraining all GCPs, (2) constraining PPK photo positions using the camera calibration model, (3) shifting PPK photo positions using the camera calibration model to selected GCPs.

- When processing PPK data, a camera calibration model must be applied.
- Delivery to the Mapping Unit shall consist of LAS file(s), the current industry-standard binary format for point-cloud data.

Documentation

Results of the sUAV survey shall be documented and included in the Final Note Package as follows (refer to Chapter 13 - Preparation of Field Note Package):

- Horizontal and vertical control procedure followed in establishing GCPs and QC check-points
- Table showing GCP/QC check-point residuals and root mean square errors (RMSE)
- Narrative describing procedures followed, including flying height, flight time, photo overlap, atmospheric conditions, etc.

Approved Uses - Camera-Equipped sUAV Surveys

General Purpose Surveys: Camera-equipped sUAV may be used on surveys which require horizontal accuracies of ≥ 0.15 feet and vertical accuracies of ≥ 0.10 feet, provided that limiting conditions described above are not present or are appropriately mitigated.

Engineering Design Surveys: The inability to accurately locate breaklines, such as top of curb, flowline, etc. and the inability to consistently segregate adjacent features which lack significant color contrast precludes the use of camera-equipped sUAV on topographic surveys for engineering design purposes at this time. sUAV may however be used to collect supplemental topographic data, for example features and terrain falling within private property adjacent to a roadway or flood control facility.

Scour Study Surveys: Scour study surveys may be conducted using camera-equipped sUAV, provided features which are submerged or obscured by foliage are captured conventionally and merged with the data collected by the sUAV.

Volumetric Surveys: Surveys made for the purpose of computing volumes of stockpiles or earthwork removals may be conducted using camera-equipped sUAV.

Additional Note: When conducting a survey which presents field personnel with a one-time access (e.g. an earthwork removal on an active construction site), additional measures shall be undertaken to ensure successful processing of the flight data. The measures taken will be at the discretion of the flight PIC, and may include on-site post-processing of GNSS data (for data validation), setting additional GCPs and check-points, etc.

Approved Uses - LiDAR-Equipped sUAV Surveys

OC Survey has yet to perform rigorous testing of LiDAR-equipped sUAV. After testing and analysis have been completed, this document will be updated accordingly.

Appendix A - OC Survey sUAV Preflight Check List

Before any sUAV flight, the following procedures must be followed:

1. When arriving at the flight location, the entire sUAV Team will visibly survey the area and discuss current conditions, including but not limited to:
   - Trees, power lines, telephone poles or other obstructions
   - People, cars, heavy equipment
   - Weather conditions: wind, visibility, sun location

This information will be used to determine whether the flight can be performed safely.

2. When preparing the sUAV for flight:
   - Inspect all components of the sUAV; look for any possible damaged or compromised parts
   - Inspect batteries for any deformation or leakage
   - Clean iPad screens

3. The compass **must** be calibrated before every flight - do not ignore this step

4. When powering sUAV, ensure that:
   - All controls are working properly
   - GNSS positioning is enabled
   - The sUAV is functioning as it should
   - Tthe take-off/landing area is clear and safe

5. Before takeoff (using iPAD App), ensure that:
   - The "HOME" point is set and plotted correctly on the map
   - Available battery power is sufficient for the flight
   - The GPS is working properly
   - There are NO warnings – warnings are not to be ignored!

CHAPTER 11
MONUMENT PRESERVATION SURVEYS

Policy Statement

Any survey which involves monument preservation shall conform to the specifications as defined in this document.

General Statement

The following document outlines the procedures to be followed when performing monument preservation surveys, and is to be used in conjunction with the **California Business and Professions Code (Land Surveyors' Act)** as cited below:

8771 Setting of monuments in general; monument perpetuation

(b) When monuments exist that control the location of subdivisions, tracts, boundaries, roads, streets, or highways, or provide horizontal or vertical survey control, the monuments shall be located and referenced by or under the direction of a licensed land surveyor or licensed civil engineer legally authorized to practice land surveying prior to the time when any streets, highways, other rights-of-way, or easements are improved, constructed, reconstructed, maintained, resurfaced, or relocated, and a corner record or record of survey of the references shall be filed with the county surveyor.

(c) A permanent monument shall be reset in the surface of the new construction or a witness monument or monuments set to perpetuate the location if any monument could be destroyed, damaged, covered, disturbed, or otherwise obliterated, and a corner record or record of survey shall be filed with the county surveyor prior to the recording of a certificate of completion for the project…

Pre-Construction Procedures

Prior to the commencement of construction activities, existing monuments shall be referenced (tied out) following the procedures outlined below:

Horizontal Control Network:

The horizontal control network may be established by GNSS (Static or RTK), conventional traverse, or a combination thereof, provided GNSS and traverse procedures are in compliance with Chapter 1 – Static GNSS, Chapter 2 – RTK GNSS, and Chapter 5 – Boundary Surveys. Specific policy relevant to the field survey shall include, but are not limited to the following points: (cont.)

- Monuments positioned by GNSS must adhere to minimum spacing requirements as defined in Chapters 1 and 2.
- All roadway monuments, boundary monuments, reference points (“tie points”), and control points must be either traversed through or double determined, in compliance with Chapter 5. Note: The “double backsight” method as defined in Chapter 5 may only be employed when use of the double determination method would result in a decrease in the strength-of-figure. In these instances, the instrument setup must be broken-down and re-erected between measurements.
- Found monuments which are included in the survey must be tied to at least one, but preferably two or more adjacent found monuments along the roadway or boundary line. This is strictly for the purpose of verifying the location of a monument and its reference points, not an effort to re-establish a roadway centerline or a boundary line.
- All points tied to the survey must be occupied by a tripod/tribrach assembly; with the exception of a “peanut prism” fitted with a level bubble, layout rods are not to be used.

Network Processing:

Network processing shall be performed using Star*Net (or equivalent) least squares adjustment software. The network processing style shall be at the discretion of the party chief, provided the processing is in compliance with Chapter 12 – Network Processing. Processing options follow:

- Multi Stage Adjustment: Static GNSS observations are adjusted, and these values are then fixed in subsequent adjustments containing RTK and/or conventional measurements
- Hybrid Adjustment: Static GNSS, RTK GNSS, and conventional traverse observations are adjusted simultaneously, constrained to CGPS or legacy control stations

Note: RTK observations shall not be adjusted as a stand-alone network - RTK must be combined with conventional observations in a hybrid adjustment.

Accuracy Standards:

Analysis of the network adjustment must be performed in order to ensure that OC Survey's accuracy standards have been met. All monument connections within the network must meet a minimum relative positional accuracy standard (local accuracy) of **1:10,000,** or **0.033 feet** for connection distances less than 330 feet. This computation is to be made at the 95% confidence level. In the event one or more pairs of monuments fails to meet these relative positional accuracy criteria, the network adjustment shall be reviewed and a determination made by the Senior Land Surveyor (or Project manager) as to whether or not additional observations will be made in order to improve network geometry, increase redundancy, or further isolate errors.

Pre-Construction Corner Records:

Per the ***California Business and Professions Code,*** the results of the survey shall be documented in the form of a **Corner Record (or Record of Survey)**, which is to be filed with the County Surveyor. An example of a typical Pre-Construction Corner Record can be found here. In addition to the minimum statutory requirements, pre-construction Corner Records shall include the following information:

- Indication that this is a pre-construction corner record
- Measured bearings and distances from tie points to the monument in question (Note: measured bearings and distances are derived by inverse between the final coordinate positions established by the network adjustment)
- Measured bearings and distances from the monument in question to adjacent found monuments along the roadway or boundary line
- A statement as to the origin of measured bearings, e.g. "Bearings shown hereon are assumed and are displayed for angular relationship only."
- The following statement regarding measured distances: Measured distances shown hereon are grid distances. To obtain a ground distance, multiply a grid distance by 1.0000XXXX.
- Record distances from tie points to the monument in question (if applicable)
- Corner Record numbers for adjacent found monuments along the roadway or boundary line (if applicable)
- A listing of references for all found monuments, tie points, and record distances
- A monument designation from historic records, e.g. "PI #7" (if applicable)
- Geographic locations (CCS83) of all found monuments, to be used for GIS purposes

Post-Construction Procedures

After construction activities have been completed, any monuments which have been destroyed shall be reset following the procedures outlined below:

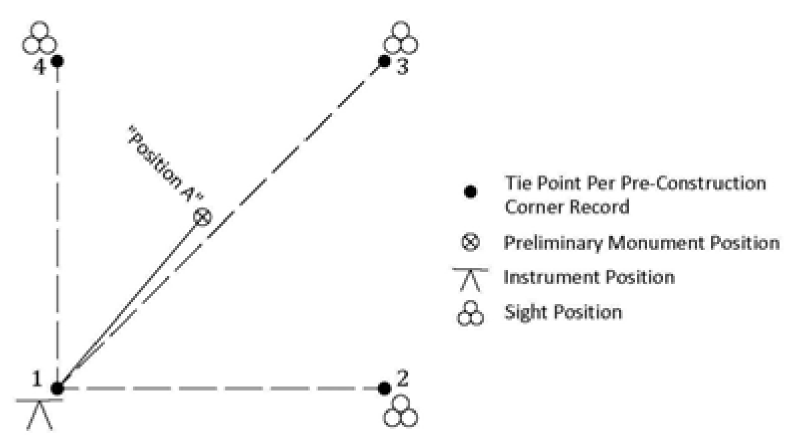
Field Procedure:

Coordinate values established by the pre-construction network adjustment are used to identify the physical position of a monument to be reset, and to verify the positions of the tie points. The example procedure detailed below represents the minimum effort necessary to ensure proper positioning of the new monument:

<u>Step 1 (See Figure 1)</u>

One of the tie points is occupied and sights are placed on the other three tie points. A backsight observation is made to one of the tie points; the backsight point and each of the remaining tie points are staked out, measured, and stored. If all computed coordinate deltas are **≤ 0.033 feet****, the monument position is staked out and marked on the ground ("Position A").

Figure 1
Establishing "Position A"



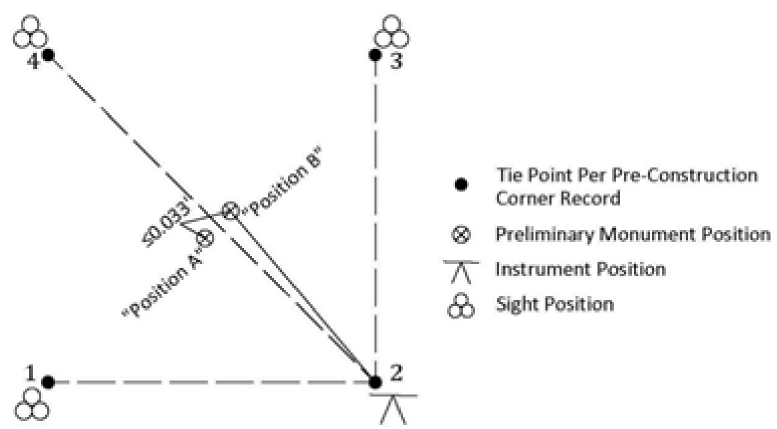
**In the event one or more of the coordinate deltas is greater than 0.033 feet, a determination must be made as to which tie point is in error. If a single tie point can be identified as the outlier, the process can be completed using the remaining three tie points and excluding the outlier. If it cannot be determined as to which point(s) are in error, all of the tie points must be re-tied to the original control network, a new network adjustment performed, and the field process repeated as outlined above.

Step 2 (See Figure 2)

After verification of all tie points in Step 1, a second tie point is occupied and sights are placed on the other three tie points. A backsight observation is made to one of the tie points; the backsight point and each of the remaining tie points are staked out, measured, and stored. If all computed coordinate deltas are **≤ 0.033 feet**, the monument position is again staked out and marked on the ground ("Position B"). A measurement is made between the two preliminary ground positions. If the measured distance is **≤ 0.033 feet**, the monument is set at the mid-point between the two preliminary positions. The position of the monument itself is then measured and stored.

Figure 2

Establishing "Position B"



Note: The procedure detailed above is to be accomplished using a total station - under no circumstances is GNSS to be used to reset a monument. A minimum of two independent total station occupations must be made to complete this process. As was required in the pre-construction survey, all points, including the monument to be reset, must be occupied with a tripod/tribrach assembly or peanut prism.

At this point, any untagged tie points which were found and referenced during the pre-construction process shall be reconstructed and tagged as required per OC Survey's *Monumentation Policy*, also shown below in the Monumentation section, under the sub-heading entitled "Reference Points (Tie Points)". The change in character of these reconstructed tie points shall be reflected on the Post-Construction Corner Record.

Any monuments which have not been destroyed by construction activities, but which are found beneath the pavement surface shall be addressed as outlined below:

- Efforts to expose a subsurface monument shall be conducted in such a manner as to be as minimally impactful as possible with regard to the integrity of the new pavement.
- When a subsurface monument is exposed, an effort shall be made to verify its position with relation to the position represented on the pre-construction Corner Record, and a determination made as to whether or not the monument has been disturbed. The position of the found monument shall be verified as follows:

  Coordinate values established by the pre-construction network adjustment are used to identify the physical positions of the found monument and the tie points. One of the tie points is occupied with a total station and sights are placed on the found monument and the other three tie points. A backsight observation is made to one of the tie points; the backsight point and each of the remaining points are staked out, measured, and stored. If all computed coordinate deltas are **≤ 0.033 feet**, the found monument's position is accepted and it is determined to be undisturbed. In the event one or more of the tie point coordinate deltas is greater than 0.033 feet, a determination must be made as to which tie point is in error.

  If a single tie point can be identified as the outlier, the process can be completed using the remaining three tie points and excluding the outlier. If it cannot be determined as to which point(s) are in error, all of the tie points must be re-tied to the original control network, a new network adjustment performed, and the field process repeated as outlined above. If the tie point coordinate deltas are within this required tolerance but the coordinate delta of the found monument is outside of tolerance, additional inspection shall be conducted to determine whether or not the found monument is to be considered disturbed.

  Note: The procedure detailed above is to be accomplished using a total station - under no circumstances is GNSS to be used to verify the position of the monument or tie points. As was required in the pre-construction survey, all points must be occupied with a tripod/tribrach assembly or peanut prism.

- When a monument is found in an undisturbed condition and lying less than 5 inches beneath the pavement surface: After verification of the monument's position (as described above), the excavated hole shall be back-filled with cold-patch asphalt. No new monument shall be set.
- When a monument is found in an undisturbed condition and lying 5 inches or more beneath the pavement surface: After verification of the monument's position (as described above), the excavated hole shall be back-filled with Quikrete® or a similar product (cold-patch asphalt is NOT to be used), and a new monument shall be set at the surface in the identical horizontal position of the found monument. This may be achieved by setting straddler or swing ties, or through use of an optical plummet. The post-construction Corner Record shall document the existence of the found monument in addition to the new monument which was set.
- When a monument is found in a disturbed condition, and field measurements confirm that its position has been compromised, the monument shall be considered destroyed.

Final Network Processing and Verification of Monument Position:
After completion of the procedures detailed above and the resetting of the monument, the process which was employed to determine the physical positioning of the monument must be documented as defined below:

- A new network adjustment is performed, processing backsight readings and stakeout data against the fixed coordinate values from the pre-construction network adjustment.
- An analysis is performed to ensure that the resultant misclosures, which are reported as either coordinate deltas or angle/distance residuals, do not exceed **0.033 feet.**
- New coordinate values are assigned to tie points which were determined to be in error, and new bearings and distances are computed and reported in the post-construction Corner Record

Post-Construction Corner Records:

Per the ***California Business and Professions Code***, the results of the survey shall be documented in the form of a **Corner Record (or Record of Survey)**, which is to be filed with the County Surveyor. An example of a typical Post-Construction Corner Record can be found here. In addition to the minimum statutory requirements, post-construction Corner Records shall include the following information:

- Indication that this is a post-construction corner record, with a reference made to the pre-construction corner record number
- Record and measured bearings and distances from tie points to the monument in question (Note: Unless an outlier is identified in Step 1 above, bearings and distances will be reported as "record and measured", using the original inverse data shown on the pre-construction corner record)
- Record and measured bearings and distances from the monument in question to adjacent found monuments along the roadway or boundary line (Note: Unless an outlier is identified in Step 1 above, bearings and distances will be reported as "record and measured", using the original inverse data shown on the pre-construction corner record)
- A statement as to the origin of bearings, e.g. "Bearings shown hereon are assumed and are displayed for angular relationship only."
- The following statement regarding distances: All distances shown hereon are grid distances. To obtain a ground distance, multiply a grid distance by 1.0000XXXX.
- Corner Record numbers for adjacent found monuments along the roadway or boundary line (if applicable)
- A listing of references for all found monuments, tie points, and record distances
- A monument designation from historic records, e.g. "PI #7" (if applicable)
- Geographic locations (CCS83) of all found and set monuments, to be used for GIS purposes

Monumentation

Monuments set during the course of a monument preservation survey shall meet the following criteria:

Boundary Corners

Monuments set at boundary corners for Tract Maps or Parcel Maps, or on any interior lot or parcel lines to be further subdivided, or for future subdivision Records of Survey:

- Monuments which fall in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall in the surface of asphalt paving shall consist of a durable spike (minimum 4 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of a 2 inch diameter iron pipe with a tag or disk.
- All tags/washers/disks referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe.

Lot and Parcel Corners

Monuments set at lot and parcel corners for Tract Maps, Parcel Maps, Records of Survey, Corner Records, Lot Line Adjustments, and Certificates of Compliance:

- Monuments which fall in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall in the surface of asphalt paving shall consist of a durable spike (minimum 4 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of a 1 inch diameter iron pipe with a tag.
- All tags/washers referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe.

Street Centerline Points

Monuments set at street intersections, the controlling points along the centerlines of streets, and where boundary lines are produced to intersect street centerlines:

- Monuments which fall in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall in the surface of asphalt paving shall consist of a durable spike (minimum 4 inches in length) with a washer. A Survey Monument Type "A" (monument well), per OC Public Works Standard Plan 1405, may be set in lieu of spike and washer described above. The number and location of Type "A" monuments shall be as directed by the County Surveyor.
- Monuments which fall in non-paved areas shall consist of a 1 inch diameter iron pipe with a tag. A Survey Monument Type "B", per OC Public Works Standard Plan 1406, may be set in lieu of iron pipe and tag described above. The number and location of Type "B" monuments shall be as directed by the County Surveyor.
- All tags/washers referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe.

Reference Points (Tie Points)

Monuments which represent tie points set for the purpose of monument perpetuation and/or preservation:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug and countersunk so as to be flush with the concrete surface.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of a 1 inch diameter iron pipe with a tag.
- All tags/washers referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe.

Any untagged tie points which are found during the course of the survey shall be reconstructed and tagged in the original location (during the post-construction phase) as follows:

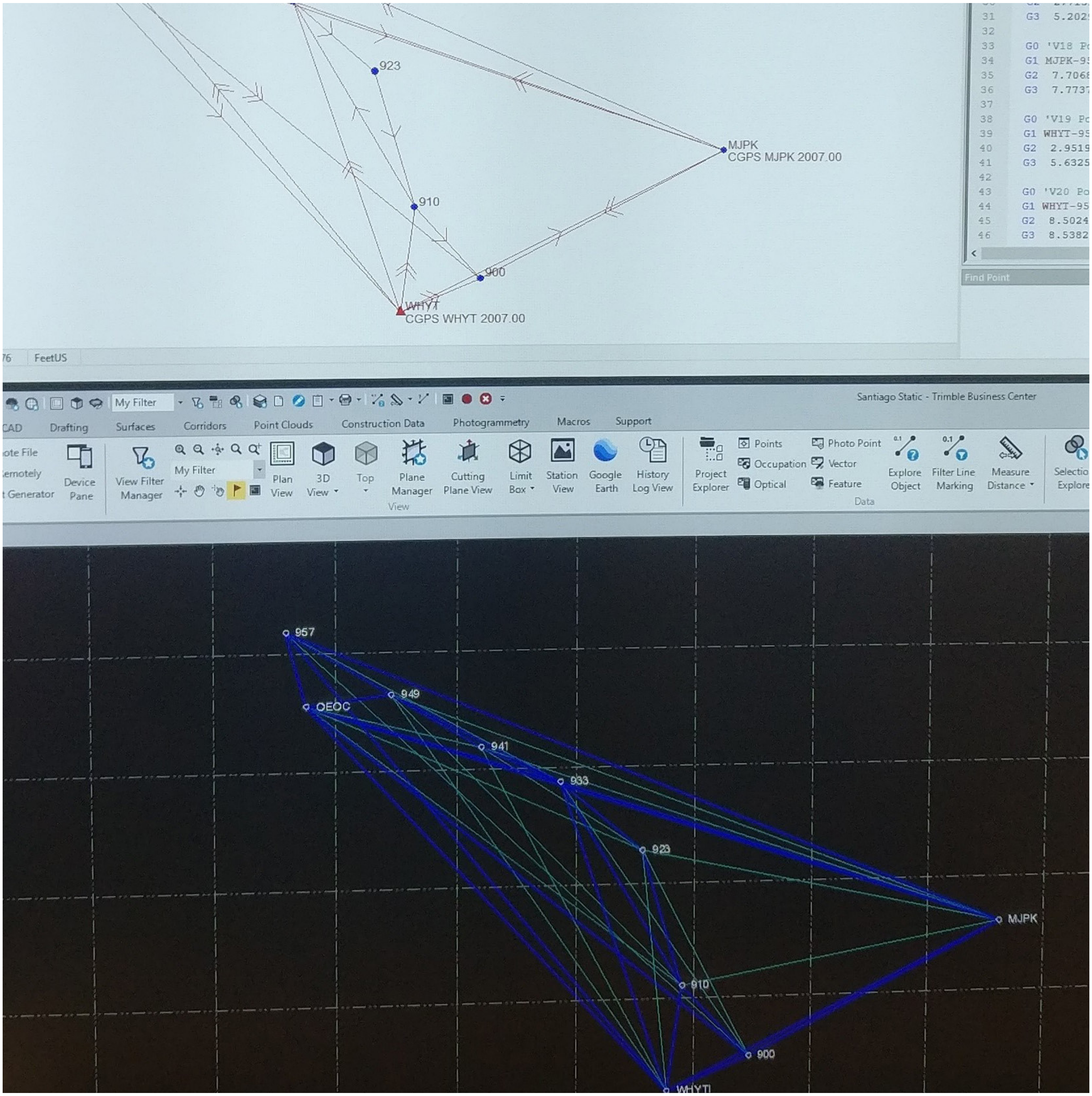
- Found lead and tack: tack is removed and a tag is secured in the existing lead plug.
- Found lead plug with hole (tack and tag missing): tag is secured in the existing lead plug.
- Found Ramset or Hilti nail: nail is removed and a tag is secured in a lead plug.
- Found chiseled cross: tag is secured in a lead plug at the center of the cross.
- Tags referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge.

Control Points

Monuments set as control points during the course of a survey:

- Monuments which fall on concrete curbs or in the surface of concrete paving shall consist of a tag secured in a lead plug.
- Monuments which fall on asphalt dikes or in the surface of asphalt paving shall consist of a spike or "MAG" nail (minimum 1-1/2 inches in length) with a washer.
- Monuments which fall in non-paved areas shall consist of an iron pipe with a tag or disk, or a rebar with an aluminum cap. Rebar must be set a minimum of 3 inches below the ground surface.
- All tags/washers/disks/caps referenced above shall be stamped with the agency name or the license number of the surveyor in responsible charge, and shall also be stamped "CP" or "CONTROL POINT".
- Tags set in iron pipes shall be of a diameter less than that of the inside diameter of the pipe. Disks affixed to iron pipes shall be of a diameter equal to that of the outside diameter of the pipe.
- Under no circumstances are plastic plugs to be used with iron pipe or rebar.

CHAPTER 12
NETWORK PROCESSING



Policy Statement

Any survey which involves the establishment of horizontal or vertical control, whether project specific or for inclusion into the OC Survey Geodetic Network, shall be adjusted by least squares adjustment software, in conformance with the specifications defined in this document.

General Statements

Star*Net is the least squares adjustment software currently employed by OC Survey. Although the procedures outlined in this document refer specifically to routines within and reports generated by Star*Net, the specifications as defined herein may be applied to other equivalent least squares adjustment programs. Contractors employed by OC Survey who are not contractually obligated to utilize Star*Net may elect to use an alternative least squares adjustment software, provided the software employed is capable of generating a network processing report which provides all of the elements contained in the section below entitled "Analysis of the Network Adjustment".

This chapter is only concerned with office processing techniques. Refer to Chapters 1-11 of this document for detailed standards with regards to collection of field data.

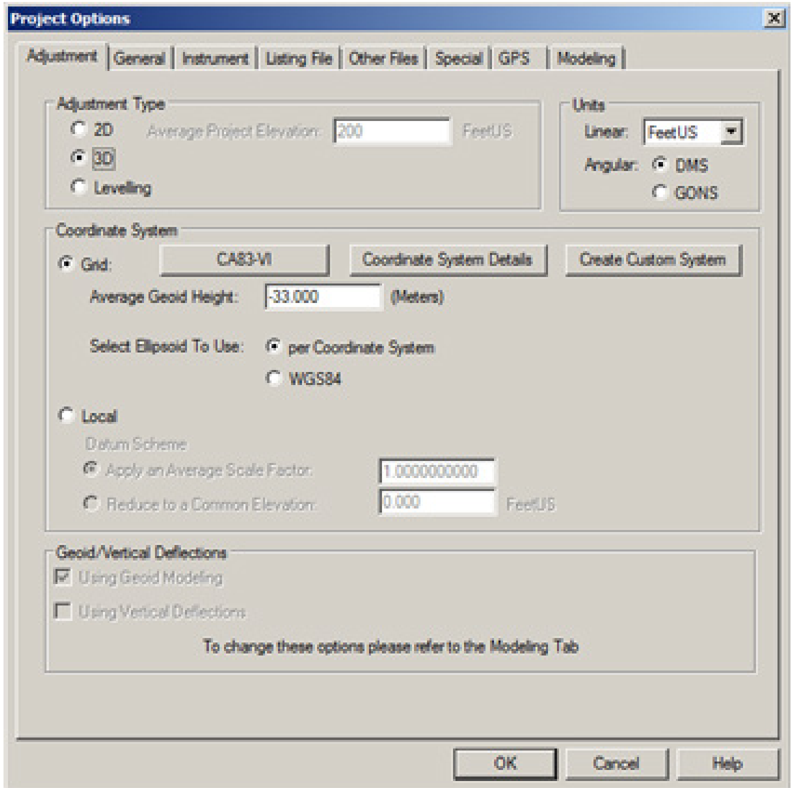
Defining Project Options

A reliable adjustment and reasonable analysis of adjustment statistics cannot be performed without first defining project options. Following is a guide to defining and understanding project options, accessed by clicking ***Options>Project***. Note that recommended settings for each tab are shown below as "Company" defaults, and are defined in a file named "***Company.def***". Once established, this file is to be shared by all users, thus ensuring that project settings are uniformly applied.

A review of each *Project Option* tab follows:

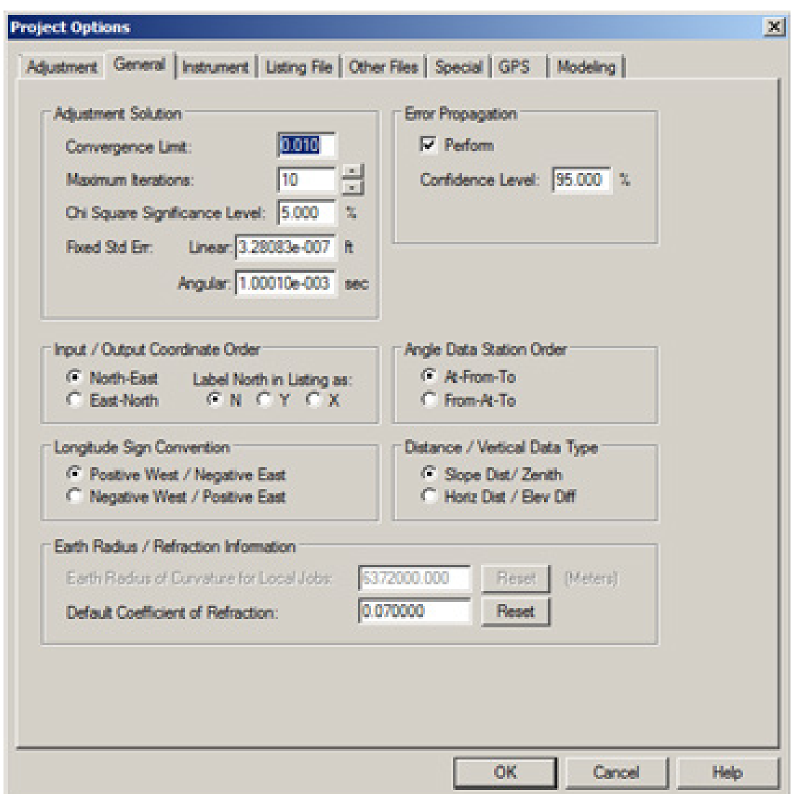
Adjustment Tab:

Select ***2D, 3D, or Leveling,*** select ***Grid Coordinate System***, and assign ***CA83-VI***. Note that a 2D project requires that an ***Average Project Elevation*** be entered so that distances can be scaled by the appropriate ellipsoid reduction factor and earth curvature corrections can be correctly applied. Be sure that ***Select Ellipsoid to Use*** is set to ***per Coordinate System.***



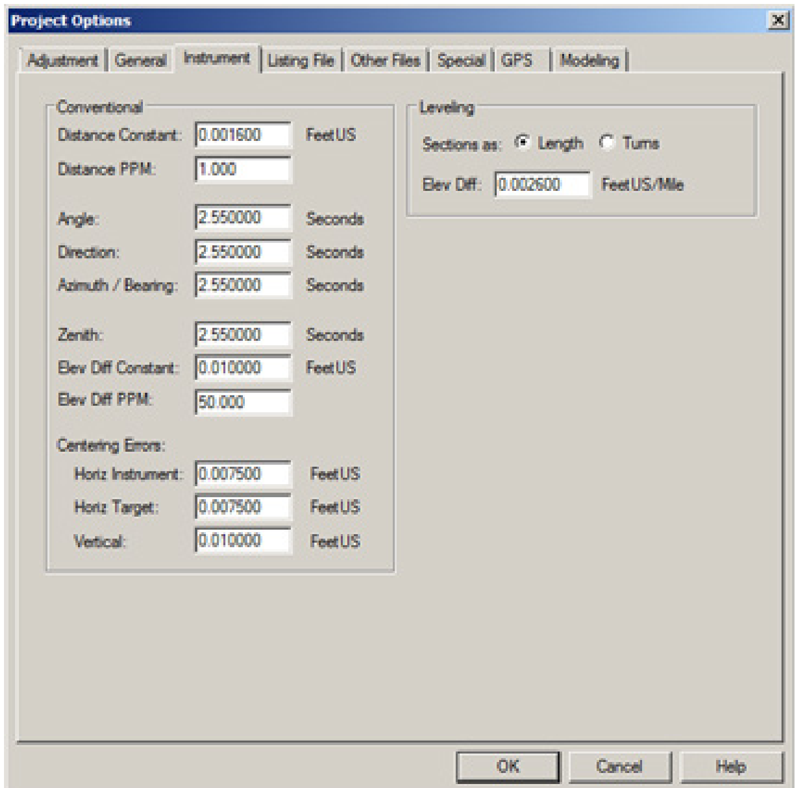
General Tab:

Values shown below in the ***Adjustment Solution*** section should not be edited. Check ***Perform Error Propagation*** and ensure that the ***Confidence Level*** is set to ***95%***.



Instrument Tab:

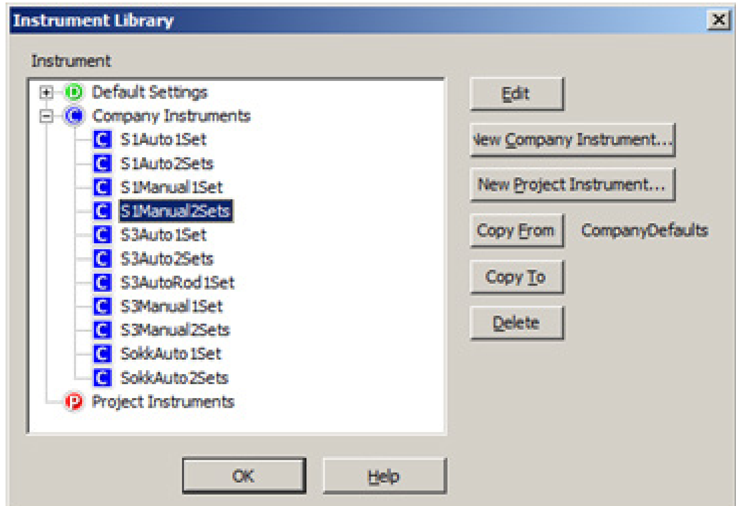
The default total station is a 3" Trimble S-Series total station, turning 2 sets of angles to tripod mounted prisms, using the Autolock feature. The default level is a Leica DNA03 level used in conjunction with a standard fiberglass bar code rod.

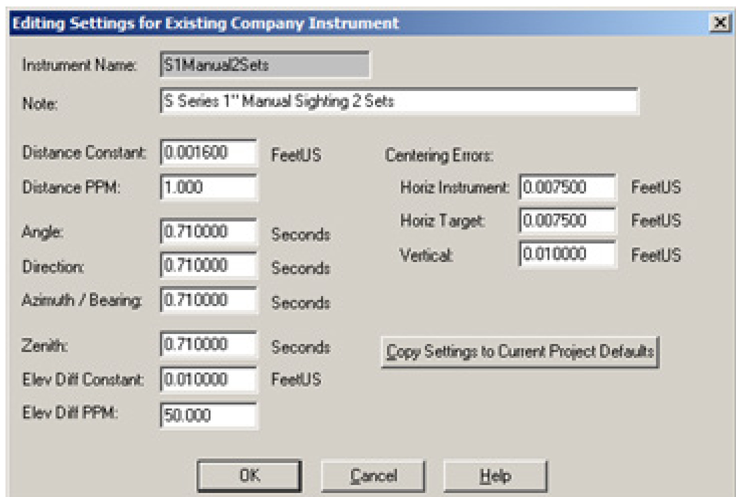


Use of a different instrument configuration is controlled by:

- Use of *Inline Options*
- Manually changing settings within the ***Instrument*** Tab
- Assigning a different instrument from the library to the current project default, which is accomplished as follows: Click ***Options>Instrument Library>Company Instruments*** and select the desired instrument; then click ***Edit***

Next click ***Copy Settings to Current Project Default*** and click ***Yes*** to verify the change.





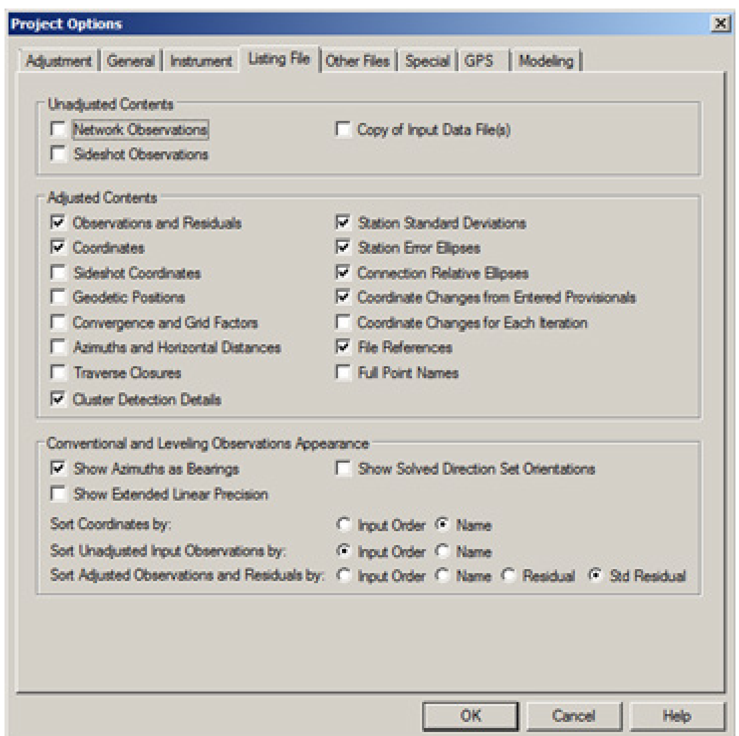
Additional Details Regarding Instrument Settings:

- Instrument configurations are a collection of standard errors calculated from manufacturer supplied instrument specifications. These specifications are calculated standard deviations and are assumed to be 1 sigma values unless otherwise stated. Trimble has verified that S Series total station standard deviation values are indeed at 1 sigma.
- Standard error = standard deviation divided by the square root of the number of observations.
- Reported angular standard deviations are for one pointing, which is the observation of a single target. An angle measurement consists of two pointings, one to a backsight target and one to a foresight target. One set of angles consists of one direct observation and one reverse observation to each target.

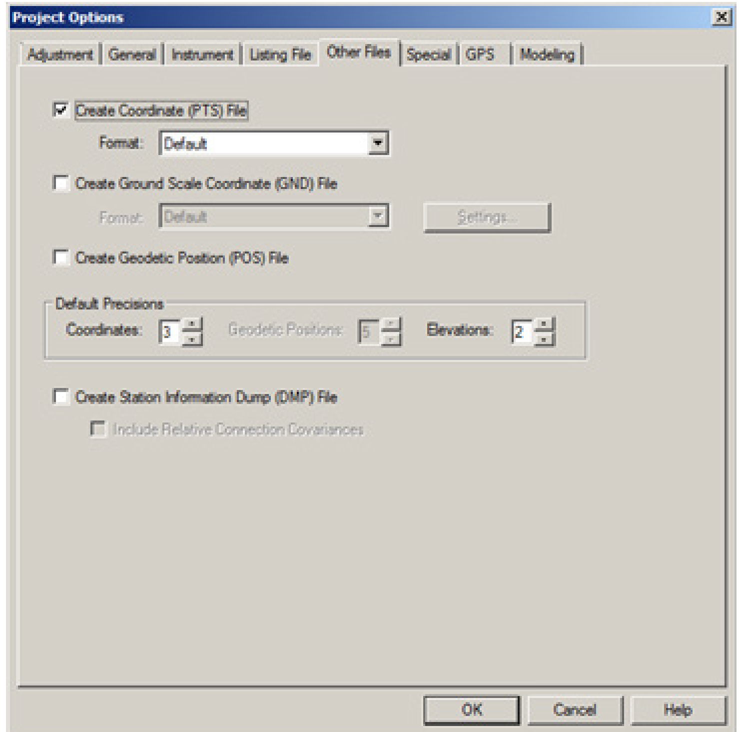
- Multiple errors within a single entity are combined by calculating the square root of the sum of the squares of the individual errors. For example, angular error when using the autolock feature consists of two components – the instrument angular error and the angular error in the autolock feature itself.
- It is important to select the correct instrument configuration from the library (or manually key in the appropriate values), as this affects weighting applied to the measurements. If multiple instrument configurations are used within a project, the appropriate instrument configuration must be referenced to each block of data through the use of the **.INST** inline option.
- As an example of the flexibility offered by use of an instrument library, the OC Survey instrument library contains an instrument configuration named "S3AutoRod1Set", which can be used in the event a layout rod/bipod is used in place of a tripod mounted target. This configuration incorporates looser target centering errors.
- Specific instrument configuration settings are compiled in Appendix "A"; formulas associated with calculation of these settings are provided in Appendix "B".

Listing File Tab:

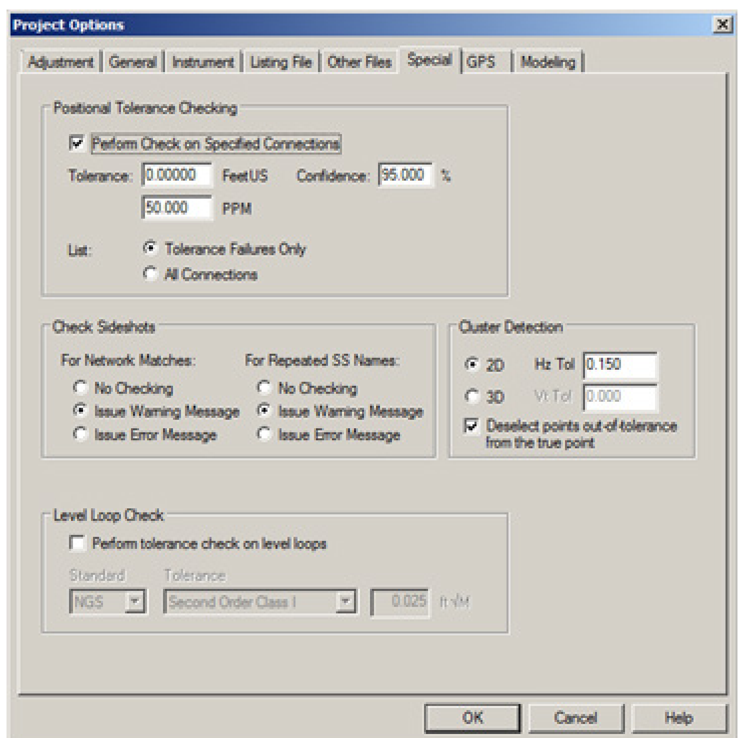
Recommended first-run options are shown below. This configuration will produce a well-organized adjustment report (**.LST** file), facilitating a logical approach to analyzing adjustment results and, if necessary, debugging input data. Additional options may be selected and included in the final report if desired, but are not necessary for analysis of adjustment results, and could tend to clutter and distract from the key elements of the report. For the final report, additional options may be selected, e.g. ***"Geodetic Positions"*** and ***"Convergence and Grid Factors".***



Other Files Tab:
These settings dictate the type and format of output coordinate files generated by the adjustment. DO NOT select ***Create Ground Scale Coordinate (GND) File***, as this will create a coordinate file which looks similar to project coordinates but has actually been converted to a scale factor 1 ground coordinate system.



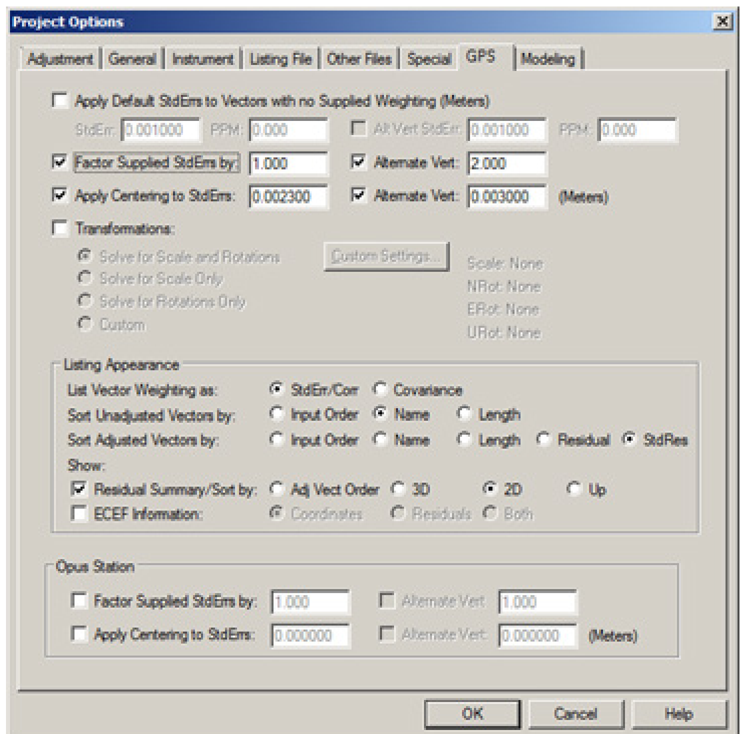
Special Tab:
Be sure to select ***Perform Check on Specified Connections***. Set the Confidence level at 95% and set the ***PPM*** to 100 for 1:10,000 relative positional accuracy, or to 50 for 1:20,000 relative positional accuracy. Relative positional accuracy, also referred to as local accuracy, is a measure of the positional accuracy relative to other points within the same network.



GPS Tab:

It is not necessary to select ***Apply Default StdErrs to Vectors with no Supplied Weighting***, as vector weighting data is included in the G2 and G3 lines output by equipment currently used by OC Survey (see sample data below).

G0 'V41 PostProcessed 31-MAY-2017 17:11:27.0 santiago static 2.asc
G1 OEOC-949   2011.530700   -732.430900   178.803900
G2 4.576122592000E-006 2.603205575700E-005 8.834203981700E-006
G3 7.169340733500E-006 -3.381548822300E-006 -1.161154159500E-005

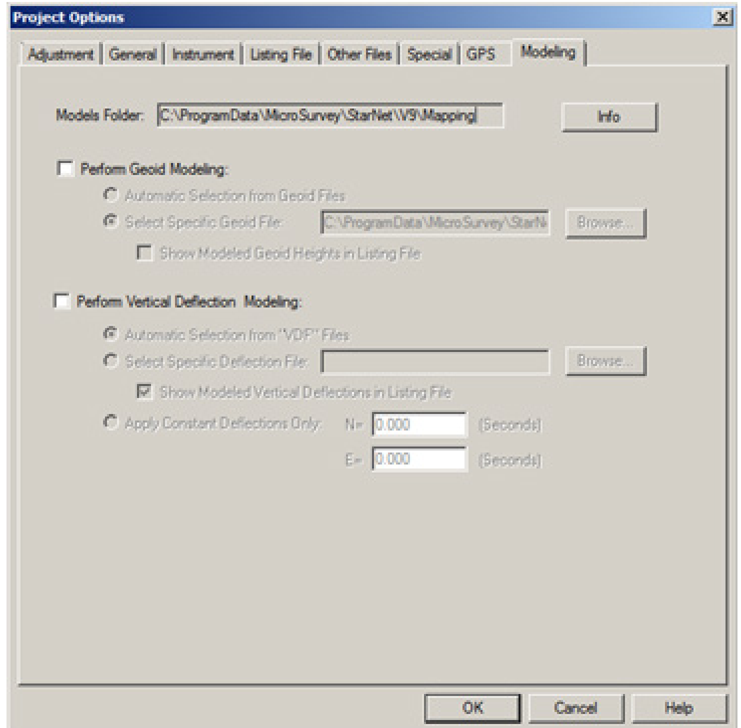


Use of the ***Factor Supplied StdErrs by*** option should not be used by default. Clean data collected with sound field procedure with well-calibrated equipment and adjusted against a reliable control network should not need to be scaled.

Select ***Apply Centering to StdErrs*** but be aware that the value entered will be applied at both ends of a vector. When processing vectors to CGPS stations, which use fixed receivers, the normal centering standard error should be divided by √2.

Modeling Tab:

If geoid modeling is required, select ***Perform Geoid Modeling*** and ***Select Specific Geoid File***, then define the path to the geoid file.



Adjustment of Vertical Control Networks

Differential leveling data requiring a simple linear adjustment need not be adjusted using least squares - for example: beginning on a fixed benchmark, leveling through project control, and closing on a second fixed benchmark. However, when multiple leveling sections or loops interconnect to form a network, points common to two or more loops shall be adjusted by least squares adjustment software.

The following procedures should be followed when processing vertical control networks.

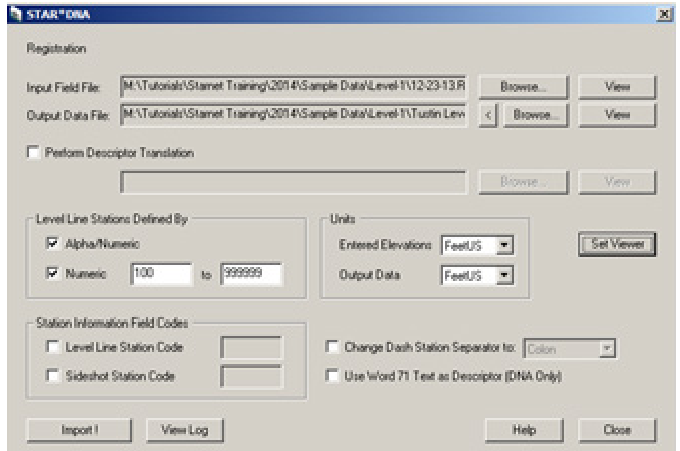
1. **Create or Open a Star*Net Project**

   Create or open a Star*Net project. As Orange County does not currently hold a license for leveling projects ***(Lev)***, select a ***Pro*** license for projects incorporating leveling data. Define ***Project Options*** as described above. Under the ***Instrument*** tab, be sure to adjust the value for ***Elev Diff*** to match the instrument and rods used in capturing the data **(see Appendix "A")**.

2. **Create a Control File**

   Create an input file ***(.DAT)*** with elevations of benchmarks to be included in the adjustment (OCS published benchmarks or control stations from an adjacent project). Add this file to the list of project ***Data Input Files.***

3. **Add Leveling Data to the Project**
   Create input files ***(.DAT)*** containing differential leveling data, either manually or through the ***STAR*DNA*** conversion routine within Star*Net, as follows: Click ***Input>STAR*DNA.*** Browse to the Leica raw data file ***(.RAW)*** and set options as desired. Click ***Import!*** Add this file to the list of project ***Data Input Files.***



4. **Adjustment – Minimally Constrained**
   Depending upon network geometry and/or redundancy, it may or may not be possible to perform a minimally constrained adjustment. An attempt should be made to assign ***fix (!)*** to as few benchmarks as possible. The remaining benchmarks are assigned ***float (*)***. This will provide a representation of how well the leveling data fits within itself. After this adjustment is run, review the resultant ***Total Error Factor.*** Ideally the error factor should be close to 1.0. An error factor less than 1.0 is an indication that assigned instrument weighting is set too loosely. An error factor greater than 1.0 is an indication that errors exceed assigned weighting. Theoretically, weighting strategies assigned in Project Options will account for systematic errors. Blunders are to be identified and removed. The remaining errors will be of a random nature, and distributed mathematically throughout the network.

   To identify blunders, a thorough analysis of the resultant ***Output>Listing*** file ***(.LST)*** is necessary. Review the ***Residuals*** within the section entitled ***"Adjusted Differential Level Observations"***. Although an error generated by a blunder will be distributed throughout the network, a larger portion of that error may be associated with leveling data within one or two leveling segments. Raw data files can be reviewed to identify possible errors due to turn imbalances, and field notes can be reviewed to identify possible point numbering blunders, but beyond those solutions it may not be possible to identify the exact source of the errors.

Once all blunders that can be identified have been corrected or removed, there may still be some leveling segments with large residuals. If the source of the large residuals cannot be identified and corrected, the data may need to be removed (using the # command), and leveling of the suspect segments would need to be repeated. After removing/replacing suspect data, rerun the adjustment and repeat the analysis. These steps are repeated until the resultant residuals and error factor are acceptable; a likely indication that all blunders have likely removed.

5. **Adjustment – Constrained**

   Once the adjustment is determined to be free of blunders and the ***Error Factor*** is within tolerance, additional benchmarks are now fixed, one by one followed by a review of the resultant error factor and residuals. If the resultant ***Error Factor*** is close to or slightly larger than it was in the minimally constrained adjustment, the adjustment is deemed to be acceptable. For the constrained adjustment, **no additional leveling segments are to be removed**, as the minimally constrained adjustment has proven the integrity of the data. Should large residuals exist, this would be an indication that the leveling data is struggling to fit the fixed benchmark elevations. At this point, a decision would have to be made as to which benchmarks are to be held.

   Note: In order to utilize the ***Network Plot*** in a Leveling project, all points included in the project must be assigned (at least approximate) horizontal positions. To accomplish this: horizontal positions of OCS Benchmarks can be determined from approximate latitude/longitude values shown on the Data Sheets; horizontal positions of project control points can be determined by first performing a 2D adjustment (see section entitled ***"Adjustment of Conventional Networks"*** below); horizontal positions of benchmarks or TBMs with no prior horizontal ties can be approximated. The Network Plot can then be exported as an exhibit in ***.DXF*** or ***.KML*** formats.

Adjustment of Static GNSS Networks

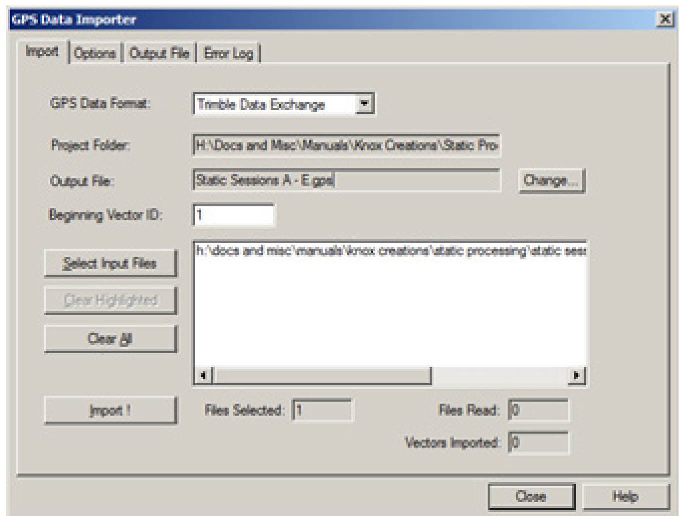
Static GNSS data may be processed as a stand-alone network, provided procedures outlined in Chapter 1 – Static GNSS are followed with regard to point spacing and network geometry. However, combining conventional traverse and differential leveling data with static GNSS vectors will sometimes result in a network with higher combined (relative) positional accuracy (see the section below entitled "Adjustment of Hybrid Networks").

The following procedures should be followed when processing stand-alone static GNSS networks.

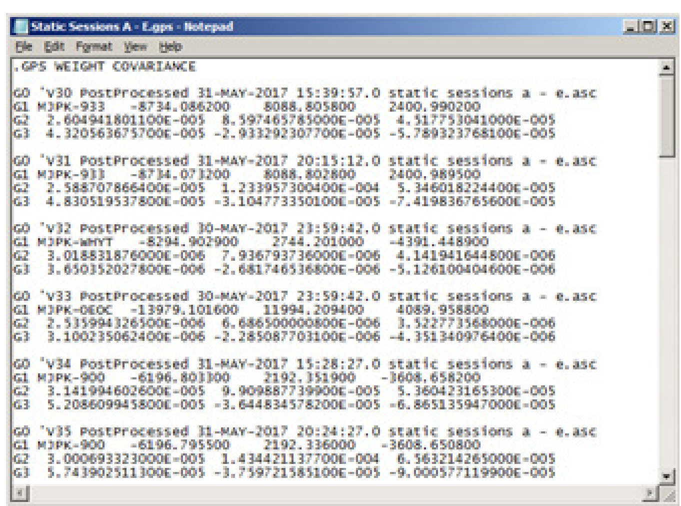
1. **Create or Open a Star*Net Project and Define Project Options**
   Create or open a Star*Net project. Be sure to select a ***Pro*** license for projects incorporating GNSS data. Define ***Project Options*** as described above.

2. **Create a Control File**
   Create an input file (***.DAT***) with positions of control stations to be included in the adjustment (CGPS stations or published legacy control stations). Include latitude, longitude, and ellipsoid height components for each control station. Add this file to the list of project ***Data Input Files.***

3. **Import Processed Baselines into the Star*Net Project**
   At this point, ***.ASC*** file(s) containing baselines which were previously processed in Trimble Business Center (TBC) and exported through Trimble Data Exchange Format (TDEF) are imported. To import the baselines, click ***Input>Import GPS Data***. Click ***Change*** to name the output file. Click ***Select Input Files*** and navigate to the ***.ASC*** file(s).



The resultant file will look like this:



This **.GPS** file will be automatically added to the list of project **Data Input Files.**

4. Adjustment – Minimally Constrained

The first step in the adjustment process is to perform a minimally constrained adjustment. A single control station is selected to be assigned ***fix (!)*** in the adjustment (x, y, and z components). The remaining control stations are assigned ***float (*)***. This will provide a representation of how well the data fits within itself. After this adjustment is run, review the resultant ***Total Error Factor***. Ideally, the resultant Total Error Factor should be close to 1.0. An error factor less than 1.0 is an indication that assigned instrument weighting and/or centering errors are set too loosely. An error factor greater than 1.0 is an indication that errors exceed assigned weighting. Theoretically, weighting strategies assigned in ***Project Options*** will account for systematic errors. Blunders are to be identified and removed. The remaining errors will be of a random nature, and distributed mathematically throughout the network.

To identify blunders, a thorough analysis of the resultant ***Output>Listing*** file ***(.LST)*** is necessary. Review the section entitled ***"Adjusted Observations and Residuals".*** Although an error generated by a blunder will be distributed throughout the network, a larger portion of that error may be associated with vectors to or from one or two points. Set-up sheets can be reviewed to verify receiver heights and point numbers, with edits made as required.

Once all blunders that can be identified have been corrected or removed, there may still be some GNSS vectors with large residuals, perhaps due to incorrect antenna heights (unverifiable) or receivers not set up accurately over the point.

If the source of the large residuals cannot be identified and corrected, the baselines may need to be removed (using the # command). After removing baselines, rerun the adjustment and repeat the analysis.

These steps are repeated until the resultant residuals and Error Factors are acceptable, a likely indication that all blunders have been removed. Be careful not to remove too many baselines! This would be an indication of bad observations, and the only remedy would be to perform additional observations.

Once satisfied with the minimally constrained adjustment, review the section of the ***.LST*** file entitled ***"Coordinate Changes from Entered Provisionals":***

Coordinate Changes from Entered Provisionals (FeetUS)

Station	dN	dE	dZ
MJPK	-0.0474	0.0078	-0.0001
OEOC	-0.0000	-0.0000	-0.0000
WHYT	-0.0177	-0.0195	-0.0349

Large coordinate changes may be an indication of poor network geometry, unidentified blunders, control stations that do not fit well in relation to others, or erroneous coordinates. In this example, station OEOC was fixed in the minimally constrained adjustment. Stations WHYT and MJPK were allowed to float, with the magnitude of the coordinate changes shown above. Ideally the two stations which are in closest agreement are fixed in the final adjustment and subsequently become the project ***Basis of Bearings.***

Note: Before running the final adjustment, rename the .LST file from the minimally constrained adjustment and save it for future reference.

5. Adjustment – Constrained

Once the adjustment is determined to be free of blunders and Error Factors are within tolerance, additional control stations are now fixed, followed by a review of the resultant error factors and residuals. If the Error Factors (in the Adjustment Statistical Summary) are close to or slightly larger than they were in the minimally constrained adjustment, the adjustment is deemed to be acceptable. For the constrained adjustment, no additional GNSS baselines are to be removed, as the minimally constrained adjustment has proven the integrity of the baselines. Should large residuals exist, this would be an indication that the GNSS baselines are struggling to fit the fixed control stations. At this point, a decision would have to be made as to which control stations are to be held. Ideally the two control stations which are in closest agreement are to be fixed in the final adjustment and subsequently become the project Basis of Bearings. In this example, OEOC and WHYT will both be fixed for the latitude and longitude and OEOC will be fixed for the ellipsoid height (only one height needs to be fixed).

Adjustment of RTK GNSS Data

Regardless of whether RTK GNSS data is collected using the CRTN or with a Base/Rover configuration, the data is adjusted with least squares by constraining to existing passive control stations (either published legacy control or existing project control). The result is similar to a Site Calibration. The key difference is that a Site Calibration performs a scale/rotation/shift, creating a **new** coordinate system based on constraint to existing control. Error in the existing control, and equally as critical, error in the RTK vectors themselves, creates an unnatural scale and rotation, in effect degrading the coordinate system and forcing this scale/rotation onto all subsequent measurements. Adjusting with least squares as described below maintains the CCS83 coordinate system by moving the base to a best-fit position, and then similarly moving subsequent measurements.

Important Note: Refer to Chapter 2 – RTK GNSS for an explanation of the limited uses of this tool. Although this section outlines the procedure for adjustment of stand-alone RTK data, it is important to recognize that RTK is generally not to be used as a stand-alone measurement tool when performing a boundary or control survey. RTK is best used to bolster a network, not define it. In order to ensure realization of acceptable combined (relative) positional accuracy standards required by these types of surveys, the network should ideally combine RTK measurements with conventional traverse and differential leveling data (see the section below entitled "Adjustment of Hybrid Networks").

The following procedures should be followed when processing stand-alone RTK GNSS data.

1. **Create or Open a Star*Net Project**
   Create or open a Star*Net project. Be sure to select a Pro license for projects incorporating GNSS data. Define Project Options as described above.

2. **Create a Control File**
   Create an input file (.DAT) with positions of passive control stations to be included in the adjustment (published legacy control or existing project control). Include northing, easting, and ellipsoid height or orthometric height (elevation) components for each control station. Project benchmark elevations may be added, where applicable. Add this file to the list of project Data Input Files.

3. **Import RTK Vectors into the Star*Net Project**
   At this point, .ASC file(s) containing vectors which were previously imported to Trimble Business Center (TBC) and exported through Trimble Data Exchange Format (TDEF)

are imported. To import the vectors, click Input>Import GPS Data. Click Change to name the output file. Click Select Input Files and navigate to the .ASC file(s). This .GPS file will be automatically added to the list of project Data Input Files.

4. **Adjustment – Minimally Constrained**

See Step 4 in the section above entitled "Adjustment of Static GNSS Networks" for procedures and logic behind the minimally constrained adjustment incorporating GNSS observations. A single control station is selected to be assigned fix (!) in the adjustment (x, y, and z components). Alternatively, for a project which ties to benchmarks or other stations with known elevations, one station can be fixed horizontally, while another station is fixed for elevation. The remaining control stations are assigned float (*).

Once satisfied with the minimally constrained adjustment, review the section of the **.LST file entitled "Coordinate Changes from Entered Provisionals":**

Coordinate Changes from Entered Provisionals (FeetUS)

Large coordinate changes may be an indication of poor network geometry, unidentified blunders, control stations that do not fit well in relation to others, or erroneous coordinates.

Station	dN	dE	dZ
RTCM0001	0.0648	-0.0125	0.1857
3258	-0.0274	0.0078	n/a
3371	-0.0000	-0.0000	n/a
3489	0.0217	0.0395	n/a
3493	0.0056	-0.0289	n/a
3D12579	n/a	n/a	-0.0000
3D12879	n/a	n/a	0.0148

In this example, RTCM0001 was the CRTN base station used for the RTK survey (position floated), 3371 was fixed horizontally, and benchmark 3D-125-79 was fixed for elevation in the minimally constrained adjustment. The other stations were allowed to float, with the magnitude of the coordinate changes shown above. If one or more of the passive control stations does not fit within desired tolerance, additional control stations should be collected. It is ideal to have control stations in at least three of the four quadrants surrounding the project area.

5. **Adjustment – Constrained**
   Once the adjustment is determined to be free of blunders and ***Error Factors*** are within tolerance, additional control stations are now fixed, one by one, followed by a review of the resultant error factors and residuals. If the **Error Factors** are close to or slightly larger than they were in the minimally constrained adjustment, the adjustment is deemed to be acceptable. For the constrained adjustment, **no additional RTK vectors are to be removed**, as the minimally constrained adjustment has proven the integrity of the vectors. For the final adjustment, a decision would have to be made as to whether to fix all points which do not degrade the network (a pseudo site calibration), or to fix two stations which are in close agreement and will subsequently become the project **Basis of Bearings**.

Adjustment of Conventional Traverse Networks

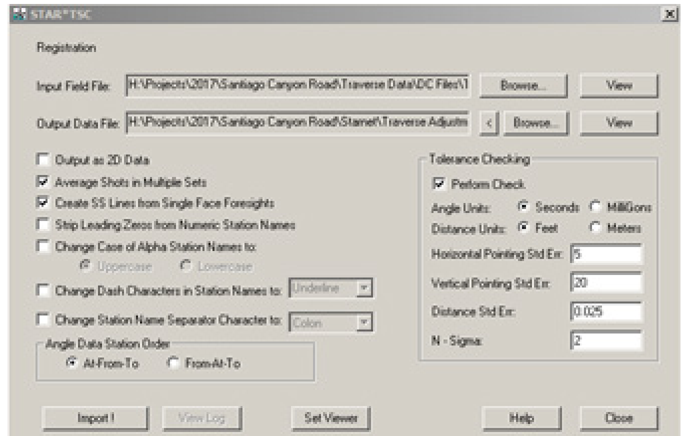
The following procedures should be followed when processing conventional traverse (total station) networks:

1. **Create or Open a Star*Net Project**
   Create or open a Star*Net project. Select a ***Std*** (or higher) license for projects incorporating conventional data. Define ***Project Options*** as described above.

2. **Create a Control File**
   Create an input file (***.DAT***) with positions of control stations to be included in the adjustment (control from previously adjusted static GNSS surveys, published legacy control stations, or control from adjacent projects). Include northing, easting, and orthometric height (elevation) components for each control station. Project benchmark elevations may be added, where applicable. Add this file to the list of project Data Input Files.

3. **Add Conventional Survey Data to the Project**
   Create input files (.DAT) containing conventional survey data, either manually, via style sheet export from the data collector or from Trimble Business Center (TBC), or through the ***STAR*TSC*** or ***STAR*JOBXML*** conversion routines within Star*Net, as follows: Click ***Input>STAR*TSC***. Browse to the data collector file (***.DC***) and set options as desired. Be sure to select ***Average Shots in Multiple Sets***. Click ***Import!*** Add this file to the list of project ***Data Input Files***. Note that by enabling ***Tolerance Checking***, erroneous or otherwise troublesome data can be identified and removed.

When working with large amounts of data (multiple crews and/or multiple days), it is advantageous to create separate **.DAT** files for each crew and/or day. This will make analysis of data and isolation of errors a more organized and efficient process.



4. **Adjustment – Minimally Constrained**

Depending upon network geometry and/or redundancy, it may or may not be possible to perform a minimally constrained adjustment. An attempt should be made to assign ***fix (!)*** to as few control stations as possible. The remaining control stations are assigned ***float (*)***. This will provide a representation of how well the data fits within itself. After this adjustment is run, review the resultant ***Error Factors***. Ideally, individual ***Error Factors (Angles, Distances,*** and ***Zeniths)*** should be close to 1.0. Error factors less than 1.0 are an indication that assigned instrument weighting and/or centering errors are set too loosely. Error factors greater than 1.0 are an indication that errors exceed assigned weighting. Theoretically, weighting strategies assigned in ***Project Options*** will account for systematic errors. Blunders are to be identified and removed. The remaining errors will be of a random nature, and distributed mathematically throughout the network.

To identify blunders, a thorough analysis of the resultant ***Output>Listing*** file ***(.LST)*** is necessary. Review the section entitled ***"Adjusted Observations and Residuals"***. Although an error generated by a blunder will be distributed throughout the network, a larger portion of that error may be associated with observations to or from one or two points. Set-up sheets can be reviewed to verify target heights and point numbers, with edits made as required. Additional clues to the source of the error may be revealed by separate review of the ***Angle, Distance,*** and ***Zenith*** residuals.

Once all blunders that can be identified have been corrected or removed, there may still be some observations with large residuals, perhaps due to (unverifiable) incorrect target heights, targets not set up accurately over the point, or poor sighting conditions. If the source of the large residuals cannot be identified and corrected, the observations may need to be removed (using the **#** command). After removing observations, rerun the adjustment and repeat the analysis.

These steps are repeated until the resultant residuals and ***Error Factors*** are acceptable, a likely indication that all blunders have been removed. Be careful not to remove too many observations! This would decrease redundancy, degrade network geometry and possibly prevent the adjustment from running. The only remedy would be to perform additional observations.

Once satisfied with the minimally constrained adjustment, review the section of the ***.LST*** file entitled ***"Coordinate Changes from Entered Provisionals".***

Coordinate Changes from Entered Provisionals (FeetUS)

Station	dN	dE
100	-0.0000	-0.0000
102	-0.0000	-0.0000
103	-0.0185	0.0042
104	0.0074	-0.0148
105	0.2510	0.1063

Large coordinate changes may be an indication of poor network geometry, unidentified blunders, control stations that do not fit well in relation to others, or erroneous coordinates. In this example, points 100 and 102 were fixed in the minimally constrained adjustment. The other stations were allowed to float, with the magnitude of the coordinate changes shown above.

5. Adjustment – Constrained

Once the adjustment is determined to be free of blunders and ***Error Factors*** are within tolerance, additional control stations are now fixed, one by one, followed by a review of the resultant error factors and residuals. If the ***Error Factors*** are close to or slightly larger than they were in the minimally constrained adjustment, the adjustment is deemed to be acceptable. For the constrained adjustment, no additional traverse observations are to be removed, as the minimally constrained adjustment has proven the integrity of the observations. Should large residuals exist, this would be an indication that the observations are struggling to fit the fixed control stations. At this point, a decision would have to be made as to which control stations are to be held.

Adjustment of Hybrid Networks - A Combination of GNSS Data (Static or RTK), Conventional Traverse Data, and Differential Leveling Data

Although GNSS data may be processed as a stand-alone network, adding conventional traverse data (total station) and differential leveling data may result in a network with higher combined (relative) positional accuracy.

The following procedures should be followed when processing hybrid networks:

1. **Create or Open a Star*Net Project**
   Create or open a Star*Net project. Be sure to select a ***Pro*** license for projects incorporating GNSS or leveling data. Define ***Project Options*** as described above.

2. **Create Control Files**
   Create an input file (***.DAT***) with positions of GNSS control stations to be included in the adjustment (CGPS stations or published legacy control stations). Include latitude, longitude, and ellipsoid height components for each control station. Create an input file with elevations of benchmarks to be included in the adjustment (OCS published benchmarks or control stations from an adjacent project). Add these files to the list of project Data Input Files.

3. **Add Survey Data to the Project**
   Create and import files (***.GPS/.DAT***) containing GNSS, conventional traverse, and/or leveling data as described above.

4. **Adjustment – Minimally Constrained**
   Before proceeding with the minimally constrained adjustment, it is prudent to first verify that the differential leveling data is free of errors and that all benchmarks to be included are deemed acceptable. Vertical control networks involving a simple linear adjustment can be verified by a review of the raw data files or Leica Level Pak Report. However, when multiple leveling sections or loops interconnect to form a network, a least squares adjustment is performed following procedures outlined in the section above entitled "Adjustment of Vertical Control Networks". Analysis of the adjustment results will serve as validation of the leveling data and identify any benchmarks which may be of questionable integrity.

After differential leveling data has been verified, the initial processing of the minimally constrained adjustment of a hybrid network can commence. At this point, only GNSS data is processed - no conventional traverse data or differential leveling data is to be included.

See Step 4 in the section above entitled "Adjustment of Static GNSS Networks" for procedures and logic behind the minimally constrained adjustment incorporating GNSS observations. A single control station is selected to be assigned ***fix (!)*** in the adjustment. The remaining control stations are assigned ***float (*)***. This will provide a representation of how well the data fits within itself. After this adjustment is run, review the resultant ***Total Error Factor.*** Ideally, the resultant ***Total Error Factor*** should be close to 1.0. An error factor less than 1.0 is an indication that assigned instrument weighting and/or centering errors are set too loosely. An error factor greater than 1.0 is an indication that errors exceed assigned weighting. Theoretically, weighting strategies assigned in ***Project Options*** will account for systematic errors. Blunders are to be identified and removed. The remaining errors will be of a random nature, and distributed mathematically throughout the network.

To identify blunders, a thorough analysis of the resultant ***Output>Listing*** file ***(.LST)*** is necessary. Review the section entitled "Adjusted Observations and Residuals". Although an error generated by a blunder will be distributed throughout the network, a larger portion of that error may be associated with vectors to or from one or two points. Set-up sheets can be reviewed to verify receiver heights and point numbers, with edits made as required.

Once all blunders that can be identified have been corrected or removed, there may still be some GNSS vectors with large residuals, perhaps due to incorrect antenna heights (unverifiable) or receivers not set up accurately over the point. If the source of the large residuals cannot be identified and corrected, the baselines may need to be removed (using the # command). After removing baselines, rerun the adjustment and repeat the analysis. These steps are repeated until the resultant residuals and ***Error Factors*** are acceptable, a likely indication that all blunders have been removed. Be careful not to remove too many baselines! This would be an indication of bad observations, and the only remedy would be to perform additional observations.

Once satisfied with the minimally constrained adjustment of the GNSS data, review the section of the ***.LST*** file entitled ***"Coordinate Changes from Entered Provisionals"***.

As stated in each of the examples given above, large coordinate changes may be an indication of poor network geometry, unidentified blunders, control stations that do not fit well in relation to others, or erroneous coordinates.

At this point, conventional traverse data (2D or 3D, see note below) and differential leveling data are added, preferably one ***.DAT*** file at a time. If the resultant Error Factors are close to or slightly larger than they were in the initial minimally constrained adjustment, the data in the ***.DAT*** files is deemed to be acceptable. Should error factors increase significantly with the addition of a ***.DAT*** file, troubleshooting is performed as described in the section above entitled "Adjustment of Conventional Traverse Networks".

Notes on inclusion of 2D vs. 3D conventional traverse data: It is important to note that although the objective in adding conventional traverse data to a GNSS network is to strengthen the network, errors in this traverse data will actually weaken the network. Even a simple target height blunder (the most common error in conventional traverse data), although seemingly just a vertical component, can degrade the horizontal component of a GNSS network. If differential leveling data is not to be included in a hybrid network, conventional traverse obseravtions are imported as 3D data. When including 3D traverse data, additional care must be given to to ensure accurate measurement of all target heights. Conversely, if leveling data is available for all points in the network, conventional traverse data should be imported as 2D data. This will significantly reduce time spent searching for blunders imbedded in the 3D data and reduce the ***Total Error Factor*** by eliminating the weakest individual component, the ***Zenith Angle Error Factor***. Also be aware that when adding 2D traverse data to a 3D project, leveling data may have to be added simultaneously in order for the adjustment to run.

5. Adjustment – Constrained

Once the adjustment is determined to be free of blunders and ***Error Factors*** are within tolerance, additional control stations are now fixed, one by one, followed by a review of the resultant error factors and residuals. If the ***Error Factors*** (in the ***Adjustment Statistical Summary***) are close to or slightly larger than they were in the minimally constrained adjustment, the adjustment is deemed to be acceptable. For the constrained adjustment, ***no additional GNSS baselines, RTK vectors, traverse observations, or leveling segments are to be removed***, as the minimally constrained adjustment has proven the integrity of the survey data. Should large residuals exist, this would be an indication that the survey data is struggling to fit the fixed horizontal and/or vertical control stations. At this point, a decision would have to be made as to which control stations are to be held. Ideally the two horizontal control stations which are in closest agreement are to be fixed in the final adjustment and subsequently become the project ***Basis of Bearings***.

Analysis of the Network Adjustment

Following are guidelines for analysis of the network adjustment, specifically the contents of the Listing File (***LST***).

1. **Adjustment Statistical Summary with Chi Square Test**

   The Adjustment Statistical Summary provides the initial glance at the overall integrity of the network adjustment.

   The first category to review is the ***Error Factor*** column. Theoretically, weighting strategies assigned in ***Project Options*** will account for systematic errors. Blunders are to be identified and removed. The remaining errors will be of a random nature, and distributed mathematically throughout the network. Ideally, with reasonable weighting strategies assigned and blunders removed, the ***Total Error Factor*** and each individual ***Error Factor (GPS Deltas, Level Data, Angles, Distances***, and ***Zeniths)*** should be close to 1.0. Error factors less than 1.0 are an indication that assigned instrument weighting and/or centering errors are set too loosely. Error factors greater than 1.0 are an indication that errors exceed assigned weighting. Refer to Steps 2 and 3 below for a strategic approach to identification and removal of errors.

   It is important to understand that Error Factors are also affected by the number of redundant observations within the network. A simple traverse around a polygon will produce very little redundancy.
   A traverse around that same figure, with forward and backward distance measurements and ties across the figure will produce a greater degree of redundancy and will ultimately lower the Error Factors as confidence in the solution is increased.

   The next item for review is the ***Chi Square Test,*** a statistical test performed during the adjustment which determines whether or not the resulting residuals are likely due to random errors. If an adjustment passes this test, it is reasonable to assume that all blunders have been removed. It is not uncommon for an adjustment to pass the Chi Square Test when minimally constrained, but fail the test once additional constraints are added (as more control stations are fixed).

This would be an indication that the survey data is struggling to fit the additional fixed control stations. If the adjustment must be constrained to these control stations, it is reasonable (though not ideal) to deem the adjustment to be acceptable, provided that significant effort is made to ensure that all identifiable errors have been removed. One negative consequence to be aware of when accepting an adjustment which fails the Chi Square Test is that the failure results in a scaling of station standard deviations and error ellipses, indicating a decreased confidence in positions and thus potential non-compliance with required accuracy standards.

Note that just because an adjustment passes the Chi Square Test and has an error factor near 1.0, an assumption cannot be made that the adjustment is error free. Detailed analysis following the steps listed below is still necessary to ensure that all identifiable errors have been removed.

2. **Adjusted Station Information**

   The Adjusted Station Information section includes a table entitled ***"Coordinate Changes from Entered Provisionals"***. "Provisionals" are coordinate values which were assigned "float". "Fixed" values are identified by changes of "0.0000". Large coordinate changes may be an indication of poor network geometry, unidentified blunders, control stations that do not fit well in relation to others, or erroneous coordinates.

3. **Adjusted Observations and Residuals**

   This section of the **.LST** file is a compilation of all adjusted observations, along with their respective **residuals**. A residual is the difference between an observed value and the corresponding adjusted value.

   Subsections include ***Adjusted GPS Vector Observations, GPS Vector Residual Summary, Adjusted Measured Angle Observations, Adjusted Measured Distance Observations, Adjusted Zenith Observations,*** and ***Adjusted Differential Level Observations.*** Although an error generated by a blunder will be distributed throughout the network, a larger portion of that error may be associated with observations to or from one or two points. Each of these subsections should be scrutinized to determine if blunders can be identified. Set-up sheets can be reviewed to verify instrument/target/receiver heights and point numbers, with edits made as required.

The column most critical to identifying suspect observations is the **Standardized Residual (StdRes)**. This is a computed ratio of a **Residual** to its corresponding **Standard Error**, or in simpler terms, a ratio of the actual magnitude of the adjustment applied to an observation to that which Star*Net expected to adjust that observation. For example: a residual of 0.022 feet (amount adjusted) with a standard error of 0.011 feet (amount expected) would result in a standardized residual of 2.0. A large value (> 3) would indicate a suspect observation.

Once all blunders that can be identified have been corrected or removed, there may still be some observations with large residuals, perhaps due to (unverifiable) incorrect target/receiver heights, targets/receivers not set up accurately over the point, or poor sighting conditions. If the source of the large residuals cannot be identified and corrected, the observations may need to be removed (using the # command). After observations are removed, the adjustment is re-run and the analysis repeated. These steps are repeated until the resultant error factors and residuals are acceptable, a likely indication that all blunders have been removed. Be careful not to remove too many observations! This would decrease redundancy, degrade network geometry and possibly prevent the adjustment from running. The only remedy would be to perform additional observations.

4. **Error Propagation**

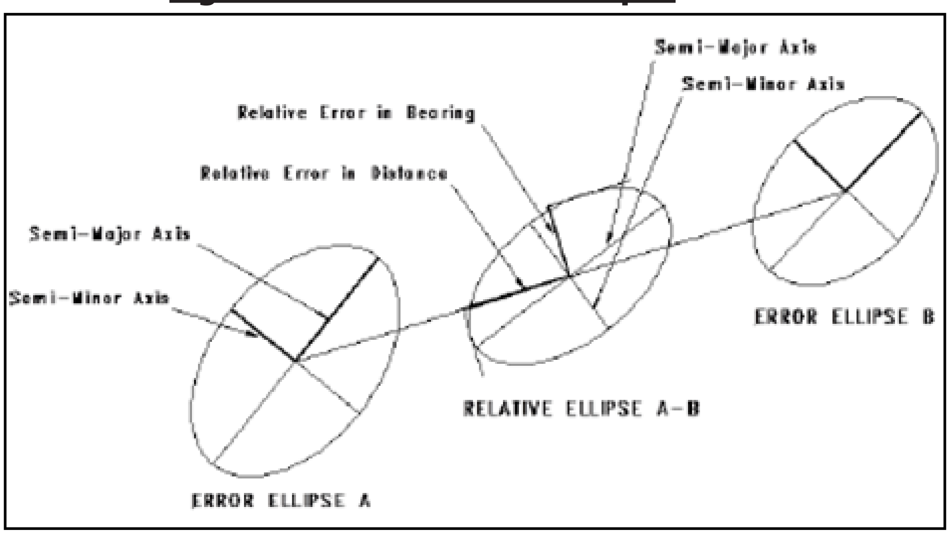
Within the context of the ***.LST*** file, Error Propagation refers to the process of determining the degree of uncertainty of the coordinate positions with respect to the datum (CCS83) and with respect to one another (connections), as computed by the least squares adjustment. A review of this section will determine whether or not the adjustment meets required accuracy standards. Refer to ***Chapters 1-11*** of this document for specific horizontal and vertical accuracy standards required for the specific type of survey being conducted.

The first section of the Error Propogation listing shows adjusted ***Station Coordinate Standard Deviations***. Standard deviations represent the amount of uncertainty in the X, Y, and Z components of each computed coordinate. Standard deviations are computed at 1 sigma (68%).

The next section shows **Station Coordinate Error Ellipses**, computed at 2 sigma (95%). The ***Semi-Major Axis*** of the ellipse is the basis of evaluation here; for practical purposes, the ellipse ***Semi-Minor Axis*** may be ignored (this is commonly accepted practice – CSRC, CLSA, et al).

Finally, ***Relative Error Ellipses*** provide an estimate of the accuracy of the relative positions of connected pairs of stations, that is, how well each pair of stations fits with each other (see Figure 1 below). This information can be more clearly illustrated by enabling a ***Positional Tolerance Check***, as shown above in the section entitled ***"Defining Project Options"***, and is one of the bases for determining whether or not the project meets required accuracy standards. As specifically stated in the **Chapters 1-3** and **Chapters 5-11** of this document, "a network adjustment shall be performed to ensure that a minimum combined (relative) positional accuracy of ***1:10,000*** (or ***1:20,000*** where required) has been achieved for all connected monument pairs." For a desired relative positional accuracy of: ***1:10,000***, the PPM value is set to ***100***; for ***1:20,000***, the PPM value is set to ***50***. This tolerance check will be applied to all pairs of connected stations, as well as additional pairs of stations manually identified in the ***DAT*** files (for example, a string of points along a boundary line). Connections of very short distances often will not meet this ***1:10,000*** standard. An alternative standard for connection distances of less than ***330 feet*** is that the ***Semi-Major Axis*** of these connection ellipses be ***≤ 0.033*** feet. In the event one or more pairs of monuments fail to pass these relative positional accuracy criteria, the network adjustment shall be reviewed and a determination made by the Senior Land Surveyor (or Project Manager) as to whether or not additional observations will made in order to improve geometry, increase redundancy, or further isolate errors.

Figure 1 – Relative Error Ellipse



Final Deliverables

Network adjustment results shall be memorialized in the form of final deliverables as described below. Refer to Chapter 13 – Preparation of the Field Note Package for directions on organization of files on the Field Survey Server.

Vertical Control Network Deliverables

- Copy of the minimally constrained .LST file
- Copy of the final .LST file
- Final coordinate file (comma or space separated text file generated by Star*Net as .PTS file)
- Spreadsheet which lists point numbers, monument descriptions, elevations, and vertical datum; distinction is made as to which benchmarks were held as fixed, including the published elevation and year leveled for each benchmark
- Adjustments which establish elevations for new benchmarks to be included in the OC Survey Geodetic Control Network will also include the following information for each new benchmark: approximate latitude and longitude (+/- 10 feet), digital photograph of the monument; digital photograph of the location; written description of the monument; detailed description of the location with to-reach instructions

Static GNSS Network Deliverables

- Copy of the minimally constrained ***.LST*** file
- Copy of the final ***.LST*** file
- Final coordinate file (comma or space separated text file generated by Star*Net as ***.PTS*** file)
- Spreadsheet (see Figure 2 below and available as hyperlink) which lists point numbers, monument descriptions, latitudes, longitudes, ellipsoid heights, combination factors, northings, eastings, orthometric heights (if applicable), horizontal datum, and vertical datum (if applicable); distinction is made as to which control stations were held as fixed, including the published coordinate values, while control stations which were floated are listed along with their provisional coordinate values and residuals (ΔN, ΔE, ΔZ)
- Statement defining the project ***Basis of Bearings***

RTK GNSS Network Deliverables

- Copy of the minimally constrained ***.LST*** file
- Copy of the final ***.LST*** file
- Final coordinate file (comma or space separated text file generated by Star*Net as ***.PTS*** file)

Figure 2 – Static GNSS Network Summary

County of Orange - OC Survey

Santiago Canyon Road - GPS Control Survey - June 2017

Starnet Adjustment "Santiago Cyn Rd" performed constraining OEOC (3D) and WHYT (2D)

Field Crew: F. Boyd & Party Starnet Adjustment: J. Knox Review: A. Andrew

Station	Latitude	Longitude	Comb Factor	North (ft)	East (ft)	Ellip Ht (ft)	N σ	E σ
900	33-40-58.007878	117-37-14.301754	0.99992595	2194806.904	6144712.657	975.405	0.010	0.010
910	33-41-59.296483	117-38-23.097191	0.99991415	2201078.392	6138982.279	1260.559	0.013	0.013
923	33-43-55.323849	117-39-04.964625	0.99992974	2212852.866	6135602.968	1013.027	0.020	0.020
933	33-44-53.131451	117-40-27.852690	0.99993816	2218790.482	6128683.057	878.444	0.010	0.009
941	33-45-22.588596	117-41-46.987411	0.99994429	2221859.779	6122042.841	772.072	0.020	0.020
949	33-46-07.090609	117-43-16.403135	0.99994838	2226463.334	6114557.476	720.143	0.013	0.013
957	33-46-58.386127	117-45-00.007901	0.99995401	2231772.518	6105886.609	642.426	0.010	0.010

Note: Standard Deviation (σ) at 1 sigma

Control Station Checks { } Indicates Fixed Value [ ] Indicates Provisional Value "*Italics*" Indicates Adjusted Value

Station	Latitude	Longitude	North (ft)	East (ft)	Ellip Ht (ft)	Δ N (ft)	Δ E (ft)	Δ Z (ft)
OEOC	**{33-45-57.075479}**	**{117-44-38.836680}**	2225550.037	6107584.607	**{1178.956}**	xxx	xxx	xxx
WHYT	**{33-40-28.152253}**	**{117-38-36.396318}**	2191881.312	6137735.654	[873.173]	xxx	xxx	-0.042
MJPK	[33-42-52.119008]	[117-33-01.643385]	*"2206066.972"*	*"6166205.295"*	[5015.470]	-0.039	0.018	-0.007

Station	Monument Descriptions
900	Set 2" IP w/OCS Disk, flush in dirt
910	Set 2" IP w/OCS Disk, flush in dirt
923	Set 2" IP w/OCS Disk, flush in dirt
933	Set 2" IP w/OCS Disk, flush in dirt
941	Set 2" IP w/OCS Disk, flush in dirt
949	Set 2" IP w/OCS Disk, flush in dirt
957	Set 2" IP w/OCS Disk, flush in dirt

Horizontal Datum = CCS83, Zone VI, OCS 2007.00 Epoch Adjustment

- Spreadsheet which lists point numbers, monument descriptions, northings, eastings, orthometric heights (if applicable), horizontal datum, and vertical datum (if applicable); distinction is made as to which control stations were held as fixed, including the published (or existing project) coordinate values, while control stations which were floated are listed along with their provisional coordinate values and residuals (ΔN, ΔE, ΔZ)
- Statement definining either the project ***Basis of Bearings*** or the basis of project control, depending on which adjustment strategy was employed

Conventional Traverse Network Deliverables

- Copy of the minimally constrained ***.LST*** file (if applicable)
- Copy of the final ***.LST*** file
- Final coordinate file (comma or space separated text file generated by Star*Net as .PTS file)

- Spreadsheet which lists point numbers, monument descriptions, northings, eastings, orthometric heights (if applicable), horizontal datum, and vertical datum (if applicable); distinction is made as to which control stations were held as fixed, including the published (or existing project) coordinate values, while control stations which were floated are listed along with their provisional coordinate values and residuals (ΔN, ΔE, ΔZ)
- Statement definining either the project ***Basis of Bearings*** or the basis of project control, depending on which adjustment strategy was employed

Hybrid Network Deliverables
- Copy of the minimally constrained ***.LST*** file
- Copy of the final ***.LST*** file
- Final coordinate file (comma or space separated text file generated by Star*Net as ***.PTS*** file)
- Spreadsheet combining the relevant elements of the adjustment spreadsheets listed above

Additional Deliverables
- Google Earth file (generated from Star*Net by clicking ***Tools>KML Exporter***)
- DXF file (generated from Star*Net by clicking ***Tools>DXF Exporter***)

Appendix "A" – Instrument Configuration Settings

Total Stations:

Trimble S-Series (3")

Manufacturer standard deviations: Angle = 3"; Autolock = 2"; Distance = 0.001m + 2 PPM

Following are 1 sigma standard errors:

1 set with manual sighting: 3.00"; 0.0023 feet; 1.4 PPM

2 sets with manual sighting: 2.12"; 0.0016 feet; 1.0 PPM

1 set with autolock: 3.61"; 0.0023 feet; 1.4 PPM

2 sets with autolock: 2.55"; 0.0016 feet; 1.0 PPM

Trimble S-Series (1”)
Manufacturer standard deviations: Angle = 1”; Autolock = 2”; Distance = 0.001m + 2 PPM
Following are 1 sigma standard errors:
1 set with manual sighting: 1.00”; 0.0023 feet; 1.4 PPM
2 sets with manual sighting: 0.71”; 0.0016 feet; 1.0 PPM
1 set with autolock: 2.24”; 0.0023 feet; 1.4 PPM
2 sets with autolock: 1.58”; 0.0016 feet; 1.0 PPM

Sokkia Net1A
Manufacturer standard deviations: Angle = 1”; Autolock = 2.5”; Distance = 0.001m + 1 PPM
Following are 1 sigma standard errors:
1 set with autolock: 2.69”; 0.0023 feet; 0.7 PPM
2 sets with autolock: 1.90”; 0.0016 feet; 0.5 PPM

Levels:
Trimble DiNi (equipped as purchased by OC Survey)
Manufacturer standard deviations: 0.0003m/2km (1km double run) with invar rods; 0.0010m/2km with standard rods
Following is the conversion to US Feet:
Invar rods: 0.0008 feet/mile
Standard rods: 0.0026 feet/mile

Leica DNA03
Manufacturer standard deviations: 0.0003m/2km (1km double run) with invar rods; 0.0010m/2km with standard rods
Following is the conversion to US Feet:
Invar rods: 0.0008 feet/mile
Standard rods: 0.0026 feet/mile

Note: Additional instrument configurations can be calculated using the formulas shown in ***Appendix “B”***.

Appendix "B" – Instrument Configuration Derivation Formulas

Total Stations:

The example calculations below use the following manufacturer published standard deviations: Angle = 3"; Autolock = 2"; Distance = 0.001m + 2 PPM

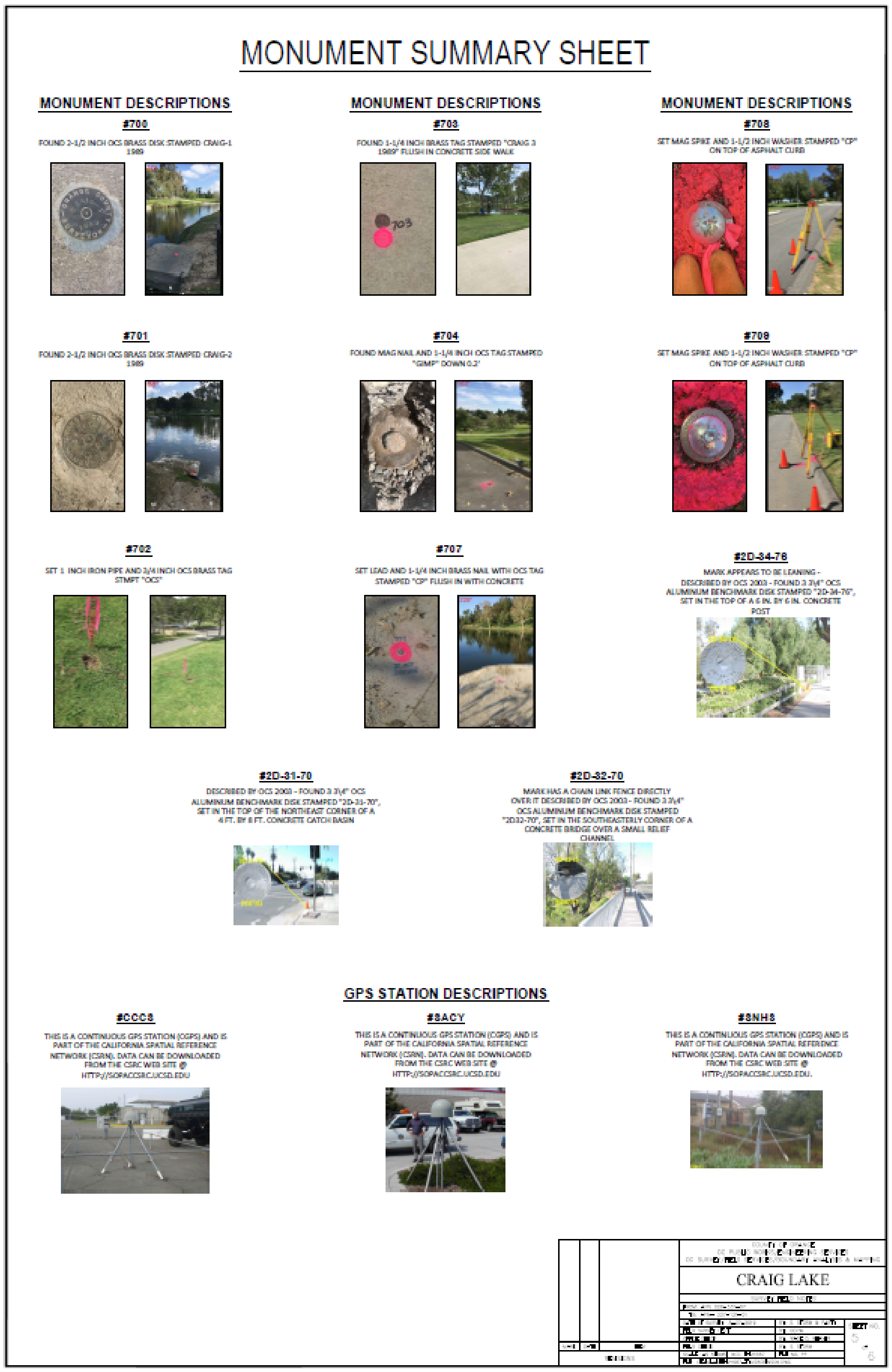
- Angular Calculation Notes: manufacturer published angular standard deviation is per pointing; each angle consists of two pointings (one to the backsight and one to the foresight); each set consists of two angles (one direct and one reverse); two sets consists of four angles (two direct and two reverse)
- Angular Standard Error with manual sighting: 3.00" for one pointing; $3.00 \times \sqrt{2} = 4.24''$ for one angle; $4.24 \div \sqrt{2} =$ **3.00" for 1 set**; $4.24 \div \sqrt{4} =$ **2.12" for 2 sets**
- Angular Standard Error with autolock function: $\sqrt{(32 + 22)} = 3.61''$ for one pointing; $3.61 \times \sqrt{2} = 5.10''$ for one angle; $5.10 \div \sqrt{2} =$ **3.61" for 1 set**; $5.10 \div \sqrt{4} =$ **2.55" for 2 sets**
- Distance Calculation Notes: manufacturer published distance standard deviation is per measurement; each set consists of two measurements (one direct and one reverse); two sets consists of four measurements (two direct and two reverse); 0.001 meters = 0.0033 feet
- Distance Standard Error: $0.0033 \div \sqrt{2} =$ **0.0023 feet for one set**; $0.0033 \div \sqrt{4} =$ **0.0016 feet for two sets**
- PPM Calculation Notes: manufacturer published PPM standard deviation is per measurement; each set consists of two measurements (one direct and one reverse); two sets consists of four measurements (two direct and two reverse)
- PPM Standard Error: 2 PPM $\div \sqrt{2} =$ **1.4 PPM for one set**; 2 PPM $\div \sqrt{4} =$ **1.0 PPM for two sets**

Levels:

The example calculations below use the following manufacturer published standard deviations: 0.0003m/1km with invar rods; 0.0010m/1km with standard rods

- $5280 \times 0.0003\text{m} \div 1000\text{m} =$ **0.0016 feet/mile with invar rods**
- $5280 \times 0.0010\text{m} \div 1000\text{m} =$ **0.0053 feet/mile with standard rods**

CHAPTER 13
PREPARATION OF THE FIELD NOTE PACKAGE



Policy Statement

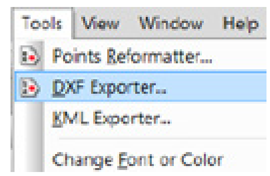
All projects initiated by a formal Survey Request shall be memorialized and archived in the format defined within this document.

Preliminary Actions

Prior to assembling the final field note package, the following files need to be generated from the Star*Net project:

DXF File:

A .DXF file can be exported from Star*Net by clicking ***Tools>DXF Exporter*** from the main menu.



This file provides a graphic representation of network connectivity and becomes the basis for the Control Establishment Sheets.

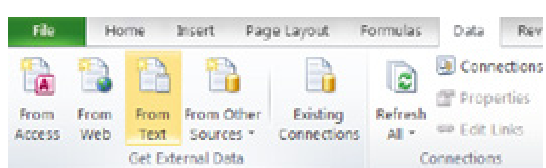
Worksheets for Conventional Observations, GPS Vector Summary, and Final Coordinates:

Excel worksheets are created for the following data sets:

- Conventional observations - extracted from the .DAT files
- GPS Vector Residual Summary - extracted from the .LST file
- Final coordinates with standard deviations (along with a note stating that coordinate standard deviations are reported at 1 sigma or 68% confidence level) - extracted from the .PTS and .LST files

Conventional observation data and the GPS Vector Residual Summary will be shown on the **Control Establishment Sheets**. The coordinate worksheet becomes a live reference for the coordinate table on the **Monument Summary Sheet**. Population of these worksheets is achieved as follows:

1. Open a blank Excel Workbook. From the main menu, click ***Data>Get External Data>From Text***.



2. Browse to the desired text file and click ***Import***. Follow steps 1-3 in the ***Text Import Wizard*** and click ***Finish***.

3. Add desired column headings and fine tune formatting as needed, then save the file.

Required Elements of the Final Field Note Package

For convenience and consistency, a template file in .DWG format is located here: **(\\lis017)\Civil 3d\OCS Notes.dwg**. Open the drawing and save it in the desired location. This file is to be used as the basis for the project Final Note Package and will contain each of the elements described below.

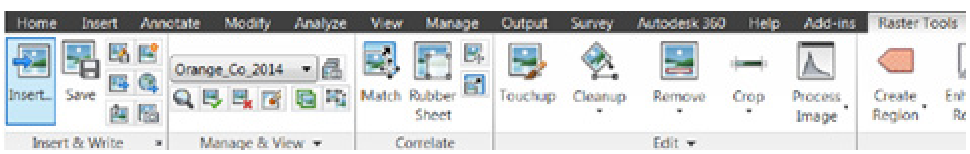
Monument Location Map / Title Sheet:

The Monument Location Map provides a plot of all monuments found and set, georeferenced to aerial imagery. Creation of this sheet is achieved as follows:

1. Import network control points and boundary points into the drawing

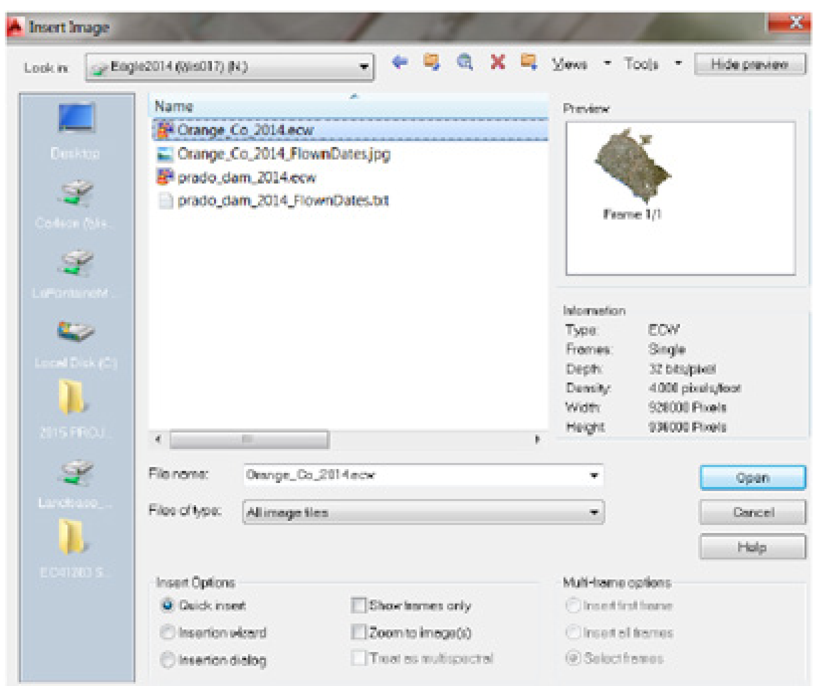
2. Add appropriate monument symbols, either by manual placement or by editing the point style. Edit point style labels to display point number only.

3. Insert the most recent Orange County Eagle Aerial imagery. **Important note:** The project coordinate system must be verified before imagery is inserted. From the ***Raster Tools*** tab, click ***Insert Image*** or type the command ***"insert"***.



From the ***Insert Image*** dialog box, select ***Insertion Wizard*** as the desired ***Insert option***. Browse to ***Orange_Co_20xx.ecw*** under the Eagle drive **(\\lis017\Eagle20xx)** and click ***Open***. Click ***Next*** through each menu, accepting the default settings, and click ***Finish***.

If the image does not appear in the correct location, verify coordinate system and project units.



Important note regarding attachment of images: Images should be loaded prior to printing and then unloaded before closing the drawing. Leaving the image loaded may cause the drawing to create a "fatal error" which will immediately close the program, and all unsaved work will be lost!

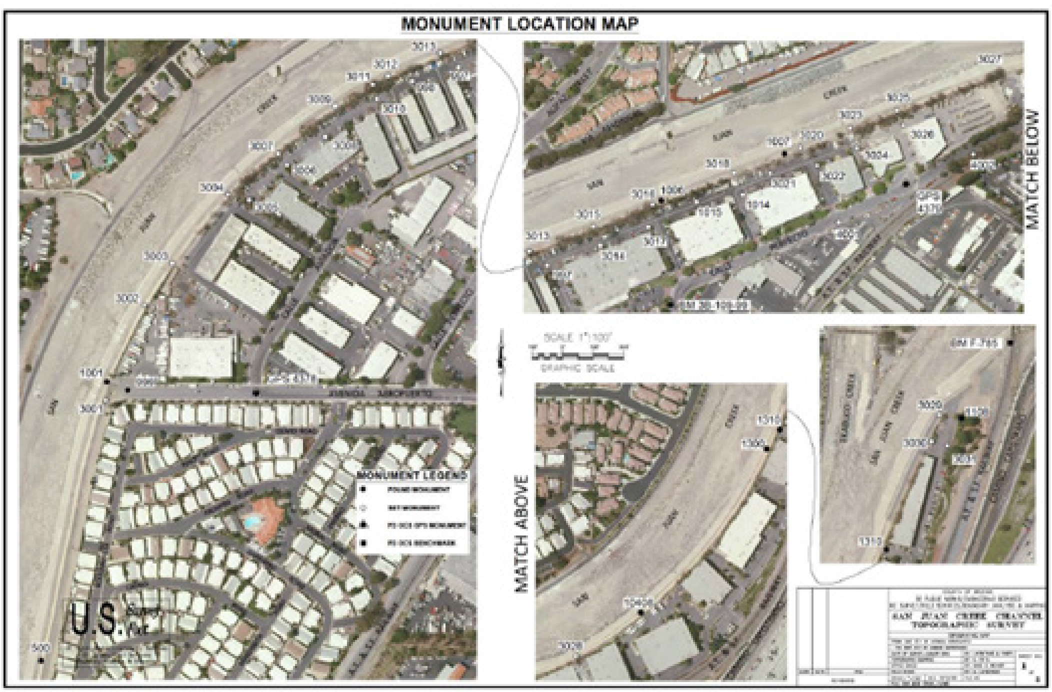
4. Place the image on a layer of its own. This will allow the transparency to be adjusted ***By Layer*** to establish a suitable contrast between image and points. If points are not visible after the image is inserted, change the ***Display Order*** of the image to ***Send to Back***.

5. After point numbers, monument symbols, and aerial image are correctly displayed, the sheets are set up in Paper Space by creating viewports in the correct layout, copying layouts if more sheets are necessary.

6. Once sheets are positioned and scaled in Paper Space, a monument legend, north arrow, scale bar, and miscellaneous text (street names, etc.) are added.

7. Fill out the ***Project Details*** table at the bottom of the sheet.

Final Sheet Appearance



PROJECT DETAILS					
Purpose of Survey:	Horizontal Datum:	Vertical Datum:	Survey Equipment:	Personnel:	Sheet Index:
				P.C.:	Sht 1:
				Instrumentman:	Sht 2:
				Rodman:	Sht 3:

Control Establishment Sheets:

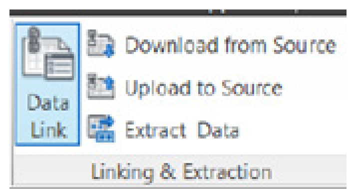
The Control Establishment Sheets provide a graphic representation of how the control points and boundary points were tied to the network. Creation of these sheets is achieved as follows:

1. Open the .DXF file exported from Star*Net, copy the linework from the .DXF, and paste into the proper layer of the new .DWG file.

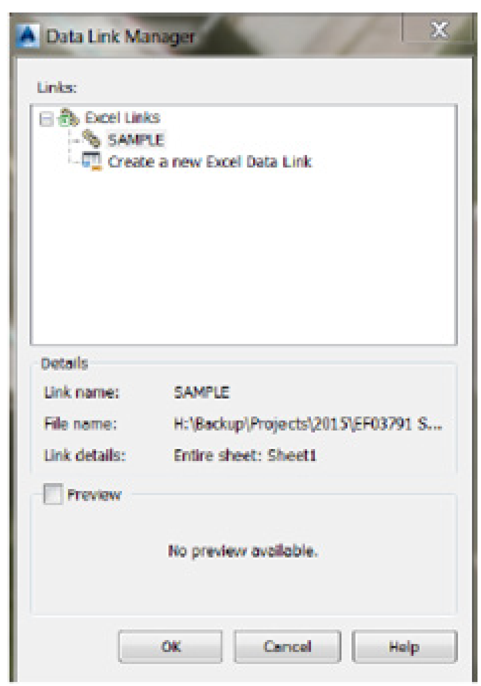
2. Set up sheets in Paper Space, displaying line connectivity, point numbers, and monument symbols. Add a monument legend and north arrow.

3. ***Establishment Notes***, a narrative outlining the control method, control constraints, etc. shall be included. If existing control was used as the basis of the survey, the process which was used to verify the network shall be documented within the narrative.

4. Insert the Excel worksheet containing observation data (created in **"Preliminary Actions"** on Page 1) as follows:

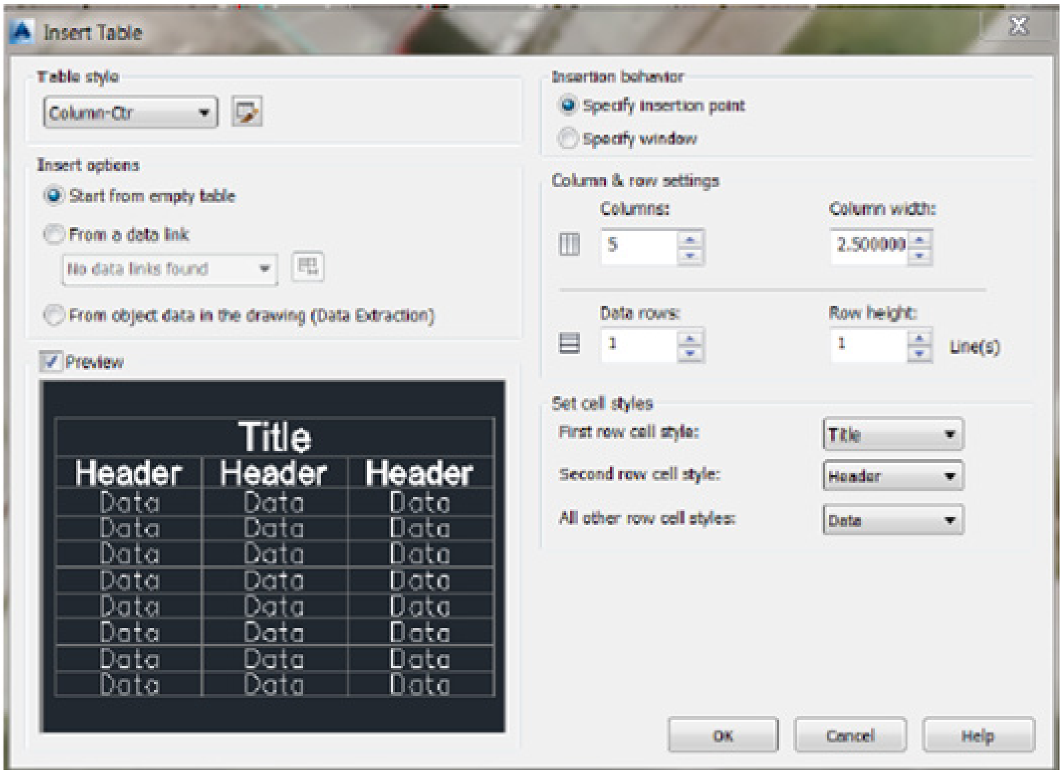


A. From the **Insert** tab click ***Linking and Extraction>Data Link***, or type the command ***"datalink".***



B. Click Create a new Excel Data Link, name the link, and click OK. Browse to the Excel file and click OK. Select the sheet to be linked and choose Link Entire Sheet, Link to a Named Range, or Link to Range. Click OK and again click OK.

C. Data is now linked and a table must be inserted: From the ***Annotate*** tab click Tables>Table or type the command ***"table".*** Specify ***Table Style***, select ***Specify Insertion Point***, select ***From a Data Link*** and choose the data link created in "step b" above. Click ***OK*** and specify insertion point on the drawing.



Entire .DAT File Imported into Excel - Unedited

ght	2016	MicroSurvey	Software	Inc.
	Field	File	:	M:\Projec File Transfer\ PACKAGE DATA\GRE BANNING MONITOR Files\EF59 TRAV1.rw5
	Processed	:	3/31/2017	7:09:13
iTo				
FROM-TO	ANGLE RT.	DIST.		
)04R1		1572.42		
)04R1-5013	151-49-10.50	1328.18	'CPT_OMIT	
104		1328.17		
104-105	293-08-55.00	386.16	'CPT_OMIT	
104-106	253-05-55.75	1378.79	'CPT_OMIT	
)13		386.18	'CPT_OMIT	
)13-103	217-56-58.50	951.53	'CPT_OMIT	
)13-102	220-01-35.50	998.94	'CPT_OMIT	
)13-101	221-53-52.00	1047.70	'CPT_OMIT	
)5		951.54	'CPT_OMIT	
)5-102	217-45-10.25	59.14	'CPT_OMIT	
)5		951.54	'CPT_OMIT	
)5-106	48-53-56.00	1474.56	'CPT_OMIT	
)5-101	217-36-12.25	118.24	'CPT_OMIT	
)3		1474.55	'CPT_OMIT	
)3-105	40-10-58.25	1111.31	'CPT_OMIT	
)3-5013	53-06-05.75	1378.78	'CPT_OMIT	
)3-107	230-46-15.00	809.27	'CPT_OMIT	
)6		809.28	'CPT_OMIT	
)6-6201	152-00-29.50	116.71	'CPT_OMIT	

Content from .DAT File to be Imported into Civil 3d

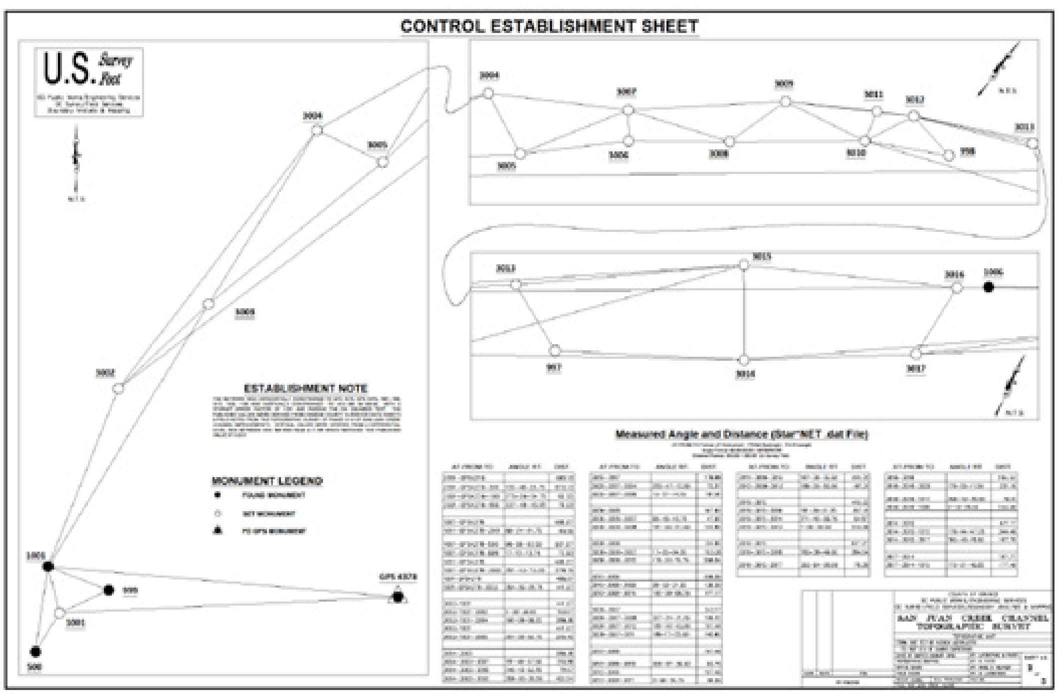
AT-FROM-TO	ANGLE RT.	DIST.
104-5004R1		1572.42
104-5004R1-5013	151-49-10.50	1328.18
5013-104		1328.17
5013-104-105	293-08-55.00	386.16
5013-104-106	253-05-55.75	1378.79
105-5013		386.18
105-5013-103	217-56-58.50	951.53
105-5013-102	220-01-35.50	998.94
105-5013-101	221-53-52.00	1047.70
103-105		951.54
103-105-102	217-45-10.25	59.14
103-105		951.54
103-105-106	48-53-56.00	1474.56
103-105-101	217-36-12.25	118.24
106-103		1474.55
106-103-105	40-10-58.25	1111.31
106-103-5013	53-06-05.75	1378.78
106-103-107	230-46-15.00	809.27
107-106		809.28
107-106-6201	152-00-29.50	116.71

GPS Vector Residual Summary

GPS Vector Residual Summary (FeetUS)
(Sorted by 2D Residual Length)

From	To	N	E	Up	2D	3D	Length
OEOC	900	-0.017	0.062	0.148	0.064	0.161	48208
MJPK	957	0.005	0.04	0.036	0.041	0.055	65723
MJPK	957	-0.005	-0.041	-0.162	0.041	0.167	65723
MJPK	933	0.012	-0.032	0.042	0.034	0.054	39843
MJPK	900	0.001	-0.033	-0.006	0.033	0.034	24602
WHYT	957	0.014	0.026	-0.089	0.029	0.093	51049
OEOC	MJPK	0.012	-0.021	-0.008	0.024	0.025	61903
OEOC	933	-0.006	0.022	0.088	0.023	0.091	22158
OEOC	900	0.02	0.004	-0.006	0.02	0.021	48208
OEOC	949	-0.011	0.014	-0.004	0.018	0.019	7048
MJPK	933	0.014	0.01	-0.062	0.017	0.065	39843
WHYT	900	-0.013	-0.01	-0.01	0.016	0.019	7567
WHYT	933	0.015	0	0.02	0.015	0.025	28393
OEOC	949	-0.002	-0.015	-0.031	0.015	0.034	7048
941	933	-0.014	0.005	-0.01	0.014	0.017	7316
949	941	-0.013	0.005	-0.017	0.014	0.022	8788
OEOC	957	0	-0.013	0.019	0.013	0.023	6473
WHYT	900	-0.002	-0.013	0.022	0.013	0.025	7567
910	933	-0.008	0.009	-0.006	0.012	0.014	20494
OEOC	957	-0.003	-0.011	0.006	0.011	0.013	6473
910	900	0.009	-0.005	-0.006	0.01	0.011	8501
WHYT	957	-0.009	-0.004	-0.086	0.01	0.087	51049
MJPK	900	0.001	0.009	-0.033	0.009	0.034	24602
OEOC	WHYT	0.003	-0.009	-0.036	0.009	0.037	45200
933	923	0.007	0.005	-0.003	0.008	0.009	9120

Final Sheet Appearance



Monument Summary Sheets:

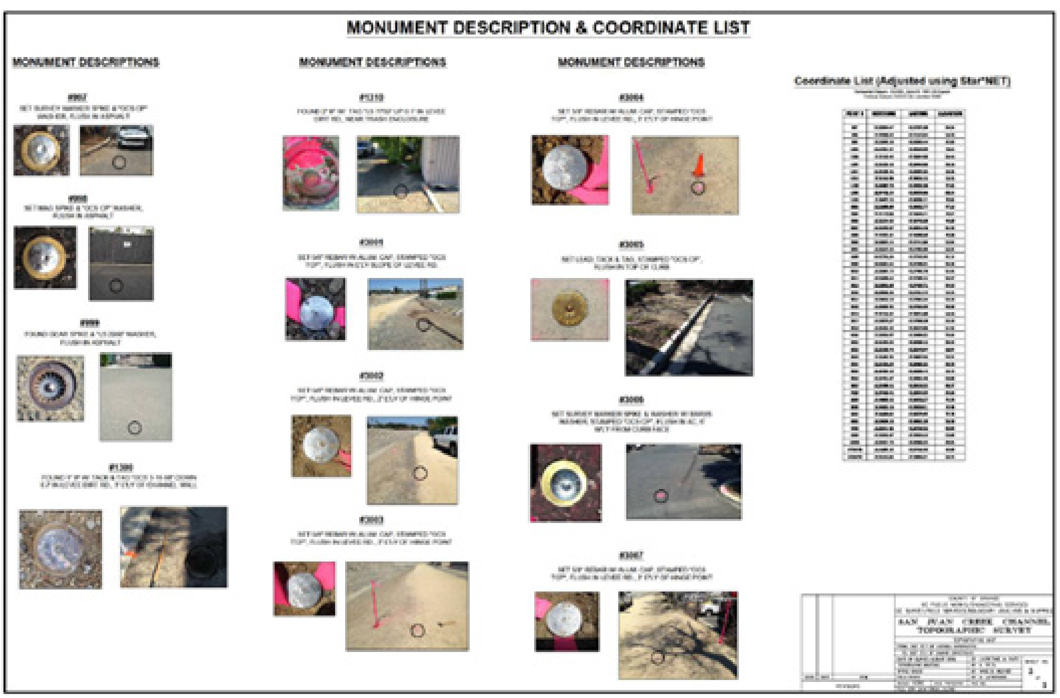
The Monument Summary Sheets compile photographs, descriptions, and coordinates of all monuments found or set during the course of the survey. Statistical data, such as coordinate standard deviations or semi-major error ellipses may also be required. Creation of these sheets is achieved as follows:

1. Set up sheets in Paper Space. Insert monument photos – quick insert method can be used, as the photos are not georeferenced.

2. Label monument photos with point numbers and descriptions – type the command "mtext" for multi-line text input.

3. Insert the Excel worksheet containing coordinate data (created in ***"Preliminary Actions"*** on Page 12-1) by creating a data link (follow the procedures outlined in ***"Control Establishment Sheets"***, steps 4a-c on Pages 12-4 through 12-6). Note: Edits to coordinate data should be made within the Excel file – once the changes are saved in Excel, Civil 3d will prompt for update.

Final Sheet Appearance

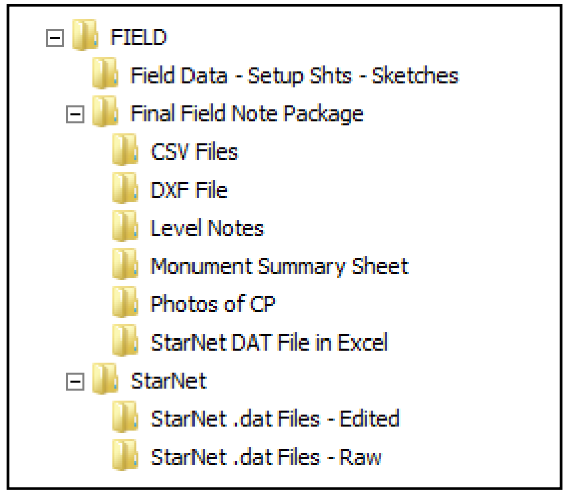


Final Product

The final product will be a single DWG file entitled ***"(project name) – Final Field Note Package.dwg"*** and a PDF file signed by the Deputy County Surveyor.

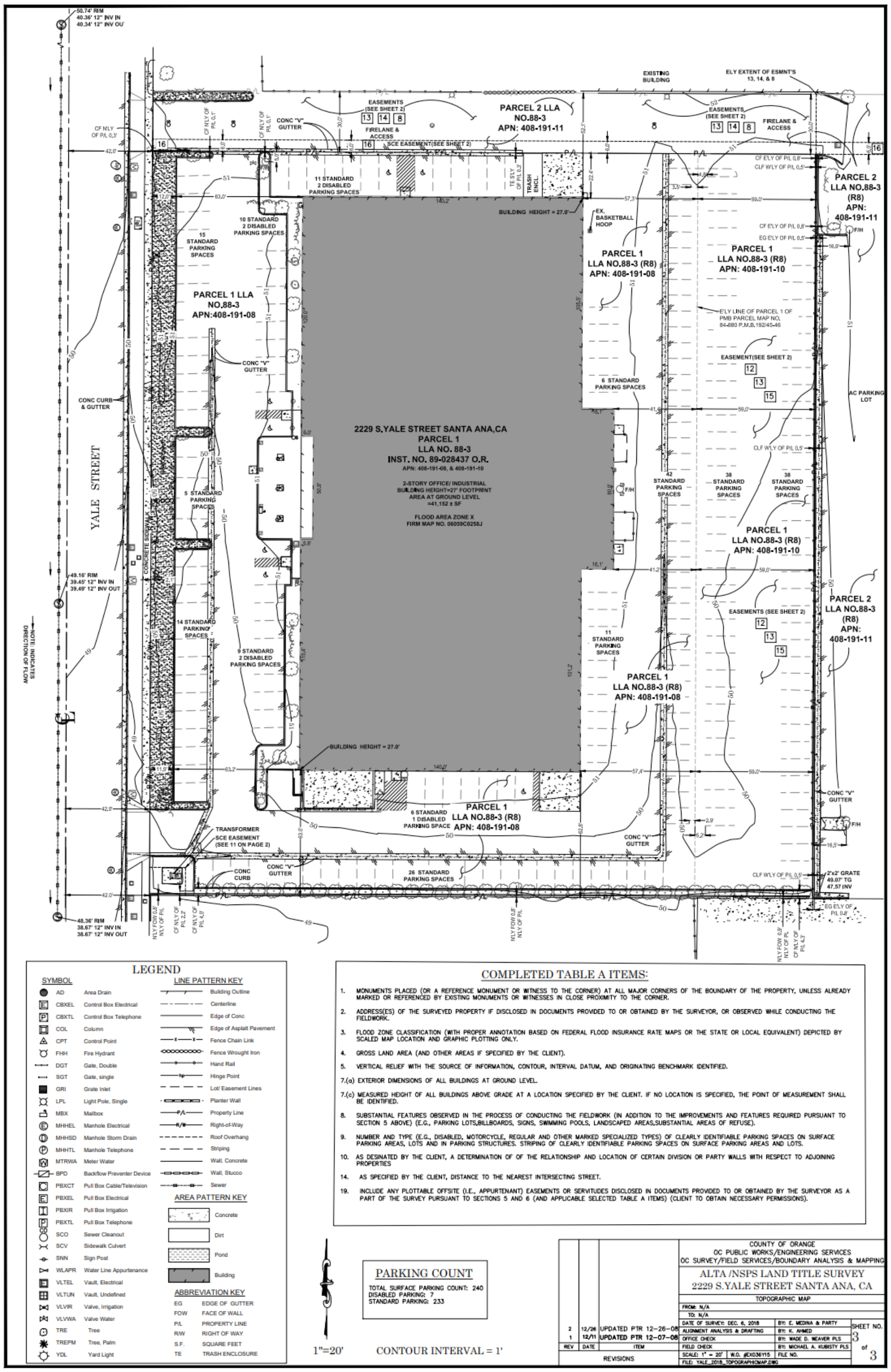
Field File Organization

Data for all projects initiated by a formal Survey Request shall be organized on the Field Survey Server as detailed here:



1. Field Data - Setup Shts - Sketches: Setup sheets; field sketches; raw files (JOB, DC, CSV attribute files, etc.)
2. Final Field Note Package:
   - (Root): Final Field Note Package DWG and signed PDF files
   - CSV Files: CSV files with final adjusted coordinates of control and boundary points in P,N,E,Z format
   - DXF File: DXF file(s) exported from Star*Net project(s) (see page 12-1 for instructions on creating DXF files)
   - Level Notes: Level notes in spreadsheet format, showing closures
   - Monument Summary Sheet: Excel spreadsheet listing coordinates and descriptions of all monuments tied to the survey, along with horizontal and vertical datum statements and a detailed narrative explaining the horizontal and vertical control strategy; points searched for and not found (SNF) should also be listed, along with a description of the search process
   - Photos of CP: Photos of control points (only necessary if the Monument App was down at the time the photos were captured); photos of the search area of points searched for and not found (SNF) should also be included
   - StarNet DAT File in Excel: Excel spreadsheet showing Star*Net DAT file data (follow steps 4a-4c on Pages 12-4 through 12-6 for instructions on creating this spreadsheet)
3. StarNet: Star*Net project files (.SNPROJ)
   - StarNet .dat Files - Edited: Topo edit logs; edited DAT files, following this required naming convention: [Abbreviated Project Name] [Task] [Date] [Crew – if more than one crew is involved] Edited.dat (e.g. Hazard Topo 8-25-18 EM Edited.dat)
   - StarNet .dat Files - Raw: Unedited DAT files, following this required naming convention: [Abbreviated Project Name] [Task] [Date] [Crew – if more than one crew is involved] Raw.dat (e.g. Hazard Topo 8-25-18 EM Raw.dat)

CHAPTER 14
MAPPING STANDARDS



Policy Statement

Any mapping performed for OC Survey Field Services, whether by a consultant or by OC Survey in-house staff, shall conform to the specifications as defined in this document.

Types of Mapping Covered by this Document

- Topographic Mapping
- Boundary Mapping
- ALTA/NSPS Mapping
- Records of Survey
- Corner Records
- Cross Sections
- Field Note Packages

Components of Topographic and Boundary Maps

Title Sheet:

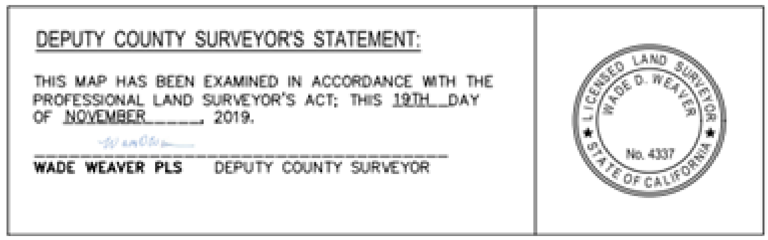
- Sheet Heading - a standard OC Public Works heading, including but not limited to: OC Public Works, Director, County Surveyor, Facility Name, Project Name, Project Limits

OC PUBLIC WORKS DEPARTMENT
SHANE SILSBY, DIRECTOR
KEVIN HILLS, COUNTY SURVEYOR
BOLSA CHICA CHANNEL (C02) PHASE I
LIMITS: LAMPSON AVE TO 783' D/S KATELLA AVE

- Location Map – a small scale depiction of the project location in relation to the nearest highways and arterial streets, along with a north arrow. This image may be generated from ESRI World Street Maps, or a similar product.

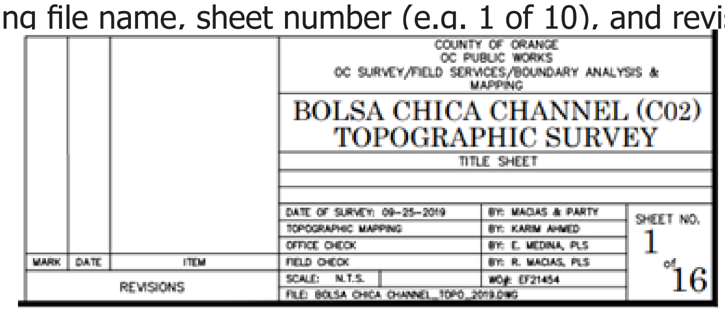
- Senior Land Surveyor's Certificate – a Senior Land Surveyor shall date, sign, and seal a statement that the survey was completed under their direction and that the survey complies with the requirements of the Professional Land Surveyors' Act

- Deputy County Surveyor's Statement – the Deputy County Surveyor shall date, sign, and seal a statement that the map has been examined in accordance with the Professional Land Surveyors' Act



- Sheet Index – a graphical display of sheet locations with aerial imagery as a background (60% transparency), along with a north arrow. Also shown in this section is a depiction of the process by which the project was tied to the California Coordinate System, including a reference to control stations used to define the project Basis of Bearings, along with grid bearings and distances between these control stations and at least two key monuments within the project. Coordinate values and monument descriptions shall be provided for all points referenced.

- Title Block - a standard OC Public Works title block containing the following info blocks: facility name (i.e. street name, flood channel name), project name, sheet type (e.g. Title Sheet, Boundary Sheet, etc.), project limits, date of survey, names of field survey party, mapping technician, office reviewer, and field checker, map scale, Work Order Number (assigned by requesting party), Survey File Number (assigned by OC Survey), drawing file name, sheet number (e.g. 1 of 10), and revisions



- Basis of Bearings Statement – a statement defining the angular orientation of the project, which shall be derived from one of the following sources:

1. CGPS Stations: Holding the computed grid bearing between two CGPS stations which are included in the SOPAC/CSRC California Real Time Network (see Example 1)
2. OC Survey Legacy GPS Monuments: Holding the coordinate values of legacy GPS monuments from published OC Survey Horizontal Control Data Sheets (see Example 2)
3. Local Monuments: Holding the record bearing between two monuments, as shown on a previously recorded map or document

Example 1

> **BASIS OF BEARINGS**
>
> THE BASIS OF BEARINGS FOR THIS SURVEY IS BASED ON THE CALIFORNIA COORDINATE SYSTEM (CCS83), ZONE VI, NAD 83, OCS (2017.50) EPOCH ADJUSTMENT, AS DETERMINED LOCALLY BY A LINE BETWEEN CONTINUOUS GLOBAL POSITIONING STATIONS (CGPS) "BLSA" AND "CCCS", BEING N66°6'41"E AS DERIVED FROM THE COORDINATES ESTABLISHED BY THE CALIFORNIA SPATIAL REFERENCE CENTER (CSRC) ALONG WITH DATASHEETS ON FILE IN THE OFFICE OF ORANGE COUNTY SURVEYOR.

Example 2

> **BASIS OF BEARINGS**
>
> THE BEARINGS SHOWN HEREON ARE BASED ON A LINE BETWEEN OCS GPS HORIZONTAL CONTROL STATIONS GPS 4149 AND GPS 4147 HAVING A BEARING NORTH 82°00'49" EAST PER RECORDS ON FILE IN THE OFFICE OF THE ORANGE COUNTY SURVEYOR.

- Horizontal Datum Statement - a statement which defines the following:

> **HORIZONTAL DATUM**
>
> COORDINATES SHOWN HEREON ARE BASED ON THE CALIFORNIA COORDINATE SYSTEM (CCS83), ZONE VI, 1983 NORTH AMERICAN DATUM (2017.50 EPOCH O.C.S. G.P.S. ADJUSTMENT).
>
> ALL DISTANCES SHOWN ARE GRID, UNLESS OTHERWISE NOTED. MULTIPLY A GRID DISTANCE BY 1.00000985 TO OBTAIN GROUND DISTANCE. THE COMBINATION FACTOR WAS HELD AT POINT NUMBER 314 N=2237883.51' E=6019333.66'
>
> ALL DISTANCES ARE BASED ON THE U.S. SURVEY FOOT.

1. Horizontal Datum (e.g. CCS83, Zone VI, 2017.50 epoch)
2. Combined Grid Factor (CF), along with a statement defining the origin of the CF. The method used in determining the CF will be dependent upon the magnitude of vertical change within the project limits and the overall size of the project. Guidelines for selection are explained below, with the decision being made by the Senior Land Surveyor:
   A. Projects with minimal vertical change: the CF is based upon a single physical point near the project center.
   B. Projects with a moderate degree of vertical change: the CF of multiple physical points distributed throughout the project are averaged.
   C. Projects with a large degree of vertical change and projects which encompass a large geographic area: multiple CF are generated, representing specific "zones" within the project.
3. Units (e.g. US Survey Foot)

- Vertical Datum Statement - a statement defining the vertical datum (e.g. NAVD88) and benchmark information (name/number, elevation, year leveled, year adjusted, and monument description)

VERTICAL DATUM

O.C.S. BM. 1B-97-99 ELEV. = 37.160' YEAR LEVELED: 2005
NAVD 1988 (O.C.S. 1995 ADJUSTMENT)

DESCRIBED BY OCS 2002- FOUND 3 3/4" OCS ALUMINUM BENCHMARK DISK STAMPED "1B-97-99" SET ON TOP OF THE NORTHWEST CORNER OF A 4 FT. BY 5 FT. CONCRETE CATCH BASIN. MONUMENT IS LOCATED IN THE NORTHWEST PORTION OF THE INTERSECTION OF KATELLA AVENUE AND VALLEY VIEW STREET, 185 FT. NORTHERLY OF THE CENTERLINE OF KATELLA AVENUE, AND 60 FT. WESTERLY OF THE CENTERLINE OF VALLEY VIEW STREET. MONUMENT IS SET LEVEL WITH THE SIDEWALK.

- Sheet Index Legend - a listing of sheet numbers and descriptions
- Purpose of Survey Statement

Alignment Map Sheet (when applicable):

- Title Block
- Map Scale
- North Arrow
- Horizontal Datum Statement
- Alignment Establishment Notes – a narrative describing the process by which the alignment was established

ALIGNMENT ESTABLISHMENT

THE CL CONSTRUCTION ALIGNMENT SHOWN HEREON WAS DERIVED BY HOLDING A BEST FIT RECORD ALIGNMENT PER RECORD PLAN C02-101-2A DATED JUNE 1959 BETWEEN THE EXISTING MID POINTS OF EXISTING CONCRETE STRUCTURES PER DIRECTIVE FROM THE CIVIL ENGINEER FOR THIS PROJECT.

SAID ALIGNMENT IS REFERENCED IN THE TOPOGRAPHIC MAP SHEETS AS "CONSTRUCTION CENTERLINE- BOLSA CHICA CHANNEL (C02)".

- Basis of Stationing – a statement defining the basis of stationing for the alignment

BASIS OF STATIONING

THE BASIS FOR STATIONING IS DERIVED FROM RECORD DRAWINGS C02-101-2A. BEGINNING AT THE MID POINT OF THE MOST SOUTHERLY EXISTING CONCRETE STRUCTURE GRADELINES WITH STATION 291+52.24 AND ENDING AT STATION 361+87.25 AS SHOWN HEREON.

- References – a listing of references related to the alignment

REFERENCES

RECORD PLAN C02-101-2A DATED JUNE 1959

Boundary Map Sheets

- Title Block
- Map Scale
- North Arrow
- Match Lines (with a scale-appropriate degree of overlap)
- Legend (including a Monument Symbol Key, a Line Pattern Key, and an Abbreviations Key)
- Street Names

- City Names and/or City Limit Lines
- Assessor's Parcel Numbers (when parcel lines are shown)
- Adjoiner Map and Document Description
- Adjoiner Parcel and/or Lot Designation
- Measured and Record Line/Curve Data (with bearings oriented north when required)
- Easement Lines (if applicable)
- Monument Symbols
- Monument Descriptions
- Establishment Symbols
- Establishment Notes
- Record Document Reference Labels
- Record Document References
- Basis of Bearings (if applicable)

Topographic Map Sheets

- Title Block
- Map Scale
- North Arrow
- Match Lines (with a scale-appropriate degree of overlap)
- Legend (including a Symbol Pattern Key, a Line Pattern Key, and an Area Pattern Key)
- Street Names
- City Names and/or City Limit Lines
- Assessor's Parcel Numbers (when parcel lines are shown)
- Residence Address
- Contour Labels (major contours always labeled; minor contours also labeled on projects with minimal vertical change)
- Contour Interval
- Pipe Inlet/Outlet Labels (pipe size and material; invert and rim elevations)
- Obscure Feature Labels (features not defined in the legend)

Examples:

An example of a typical OC Survey Topographic Survey Map can be found here. An example of a typical OC Survey Boundary Map Sheet can be found here.

Components of ALTA/NSPS Land Title Survey Maps

Note: ALTA/NSPS standards are updated on a regular basis. The most current standards can be found on the NSPS website.

Title Sheet:

- Sheet Heading - a standard OC Public Works heading, including but not limited to: OC Public Works, Director, County Surveyor, Facility Name, Project Name, Project Limits

OC PUBLIC WORKS DEPARTMENT

SHANE SILSBY, DIRECTOR

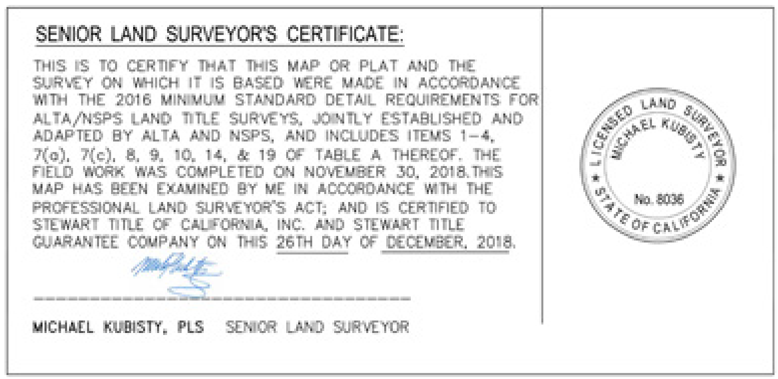
KEVIN HILLS, COUNTY SURVEYOR

ALTA/NSPS LAND TITLE SURVEY

2229 S. YALE STREET, SANTA ANA,CA

- Location Map

- Senior Land Surveyor's Certificate – a Senior Land Surveyor shall date, sign, and seal a statement that the survey was completed under their direction and that the survey complies with the requirements of the Professional Land Surveyors' Act (some content will vary by project)

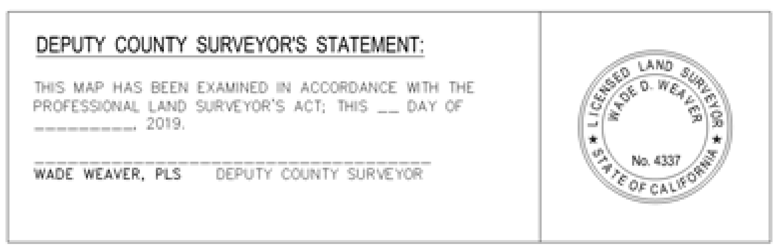

SENIOR LAND SURVEYOR'S CERTIFICATE:

THIS IS TO CERTIFY THAT THIS MAP OR PLAT AND THE SURVEY ON WHICH IT IS BASED WERE MADE IN ACCORDANCE WITH THE 2016 MINIMUM STANDARD DETAIL REQUIREMENTS FOR ALTA/NSPS LAND TITLE SURVEYS, JOINTLY ESTABLISHED AND ADAPTED BY ALTA AND NSPS, AND INCLUDES ITEMS 1-4, 7(a), 7(c), 8, 9, 10, 14, & 19 OF TABLE A THEREOF. THE FIELD WORK WAS COMPLETED ON NOVEMBER 30, 2018.THIS MAP HAS BEEN EXAMINED BY ME IN ACCORDANCE WITH THE PROFESSIONAL LAND SURVEYOR'S ACT; AND IS CERTIFIED TO STEWART TITLE OF CALIFORNIA, INC. AND STEWART TITLE GUARANTEE COMPANY ON THIS 26TH DAY OF DECEMBER, 2018.

MICHAEL KUBISTY, PLS SENIOR LAND SURVEYOR

LICENSED LAND SURVEYOR MICHAEL KUBISTY No. 8036 STATE OF CALIFORNIA

- Deputy County Surveyor's Statement – the Deputy County Surveyor shall date, sign, and seal a statement that the map has been examined in accordance with the Professional Land Surveyors' Act


DEPUTY COUNTY SURVEYOR'S STATEMENT:

THIS MAP HAS BEEN EXAMINED IN ACCORDANCE WITH THE PROFESSIONAL LAND SURVEYOR'S ACT; THIS __ DAY OF __________, 2019.

WADE WEAVER, PLS DEPUTY COUNTY SURVEYOR

LICENSED LAND SURVEYOR WADE D. WEAVER No. 4337 STATE OF CALIFORNIA

- Sheet Index – a graphical display of sheet locations with aerial imagery as a background (60% transparency), along with a north arrow. Also shown in this section is a depiction of the process by which the project was tied to the California Coordinate System, including a reference to control stations used to define the project Basis of Bearings, along with grid bearings and distances between these control stations and at least two key monuments within the project. Coordinate values and monument descriptions shall be provided for all points referenced.

- Basis of Bearings Statement - a statement defining the angular orientation of the project, which is to be derived from one of the following sources:

> **BASIS OF BEARINGS**
>
> THE BASIS OF BEARINGS FOR THIS SURVEY IS BASED ON THE CALIFORNIA COORDINATE SYSTEM (CCS83), ZONE VI, NAD 83, OCS 2007.00 EPOCH ADJUSTMENT, AS DETERMINED LOCALLY BY A LINE BETWEEN CONTINUOUS GLOBAL POSITIONING STATIONS (CGPS) "SACY" AND "CCCS", BEING NORTH 12°58'34" EAST AS DERIVED FROM THE COORDINATES PUBLISHED AND IN FILE IN THE OFFICE OF THE ORANGE COUNTY SURVEYOR.

  1. CGPS Stations: Holding the computed grid bearing between two CGPS stations which are included in the SOPAC/CSRC California Real Time Network
  2. OC Survey Legacy GPS Monuments: Holding the coordinate values of legacy GPS monuments from published OC Survey Horizontal Control Data Sheets
  3. Local Monuments: Holding the record bearing between two monuments, as shown on a previously recorded map or document

- Horizontal Datum Statement – a statement which defines:

> **HORIZONTAL DATUM**
>
> COORDINATES SHOWN HEREON ARE BASED ON THE CALIFORNIA COORDINATE SYSTEM (CCS83), ZONE VI, 1983 NORTH AMERICAN DATUM (2017.50 EPOCH O.C.S. G.P.S. ADJUSTMENT).
>
> ALL DISTANCES SHOWN ARE GRID, UNLESS OTHERWISE NOTED.
> MULTIPLY A GRID DISTANCE BY 1.00000985 TO OBTAIN GROUND DISTANCE. THE COMBINATION FACTOR WAS HELD AT POINT NUMBER 314 N=2237883.51' E=6019333.66'
>
> ALL DISTANCES ARE BASED ON THE U.S. SURVEY FOOT.

  1. Horizontal Datum (e.g. CCS83, Zone VI, 2017.50 epoch)
  2. Combined Grid Factor (CF), along with a statement defining the origin of the CF. The method used in determining the CF will be dependent upon the magnitude of vertical change within the project limits and the overall size of the project. Guidelines for selection are explained below, with the decision being made by the Senior Land Surveyor:
     - A. Projects with minimal vertical change: the CF is based upon a single physical point near the project center.
     - B. Projects with a moderate degree of vertical change: the CF of multiple physical points distributed throughout the project are averaged.
     - C. Projects with a large degree of vertical change and projects which encompass a large geographic area: multiple CF are generated, representing specific "zones" within the project.
  3. Units (e.g. US Survey Foot)

- Vertical Datum Statement – a statement defining the vertical datum (e.g. NAVD88) and benchmark information (name/number, elevation, year leveled, year adjusted, and monument description)

> **VERTICAL DATUM**
>
> O.C.S. BM. 1B-100-99 ELEV. = 173.090' YEAR LEVELED: 2010
>
> NAVD 1988 (O.C.S. 1995 ADJUSTMENT)
>
> DESCRIPTION: DESCRIBED BY OCS 2002 – FOUND 3 3/4" OCS ALUMINUM BENCHMARK DISK STAMPED "1B-100-99" SET ON TOP OF THE SOUTHEAST CORNER OF A 5 BY 8 FT. CONCRETE CATCH BASIN. MONUMENT IS LOCATED ON THE WESTERLY SIDE OF MAIN STREET, 375 FT. NORTHERLY OF THE CENTERLINE OF KATELLA AVENUE, AND 41 FT. WESTERLY OF THE CENTERLINE OF MAIN STREET. MONUMENT IS SET LEVEL WITH THE SIDEWALK.

- Flood Zone Classification – based upon the FEMA Flood Insurance Rate Map and Setback Requirements

FLOOD ZONE

FLOOD ZONE X – SHADED, DESCRIBED AS AREAS OF 0.2% ANNUAL CHANCE FLOOD; AREAS OF 1% ANNUAL CHANCE FLOOD WITH AVERAGE DEPTHS OF LESS THAN 1 FOOT OR WITH DRAINAGE AREAS LESS THAN 1 SQUARE MILE; AND AREAS PROTECTED BY LEVEES FROM 1% ANNUAL CHANCE FLOOD PER NATIONAL FLOOD INSURANCE PROGRAM (NFIP) & FLOOD INSURANCE RATE MAP INFORMATION LOCATED AT ORANGE COUNTY, CALIFORNIA AND INCORPORATED AREAS, CITY OF ORANGE; PANEL 0258J C0142J WITH AN EFFECTIVE DATE OF DECEMBER 3, 2009

- Zoning and Setback Requirements

ZONING AND SETBACK REQUIREMENTS

THE PROPERTY IS WITHIN THE "LIGHT INDUSTRIAL (M-1)" ZONE OF THE CITY OF SANTA ANA.MINIMUM REQUIRED YARD SETBACKS IN FEET FOR ZONE DISTRICT M1 (PER CITY OF SANTA ANA CODE OF ORDINANCES SECTION 41-475 – YARD REQUIREMENTS):
YARD ABUTTING ARTERIAL STREET :20' YARD ABUTTING NON-ARTERIAL STREET :10'
"THE YARD REQUIRED BY PARAGRAPH (A) SHALL INCLUDE A STRIP IMMEDIATELY ADJACENT TO THE STREET AND SHALL BE OF A WIDTH NOT LESS THAN:10 FEET, IF THE STREET IS DESIGNATED IN THE GENERAL PLAN OF THE CITY AS AN ARTERIAL STREET; OR (II) FIVE (5) FEET, IF THE STREET IS NOT SO DESIGNATED AS AN ARTERIAL STREET.(C) THE AREA OF ANY ONE (1) VEHICULAR DRIVEWAY SHALL BE CONSIDERED PART OF THE AREA OF ANY YARD REQUIRED BY PARAGRAPH (A) TO THE EXTENT THAT: (I) THE DRIVEWAY IS APPROXIMATELY PERPENDICULAR TO THE STREET; AND (II) THE DRIVEWAY DOES NOT EXCEED THIRTY (30) FEET IN WIDTH.(D) EXCEPT AS PROVIDED IN PARAGRAPH (C), ANY YARD REQUIRED BY THIS SECTION SHALL BE LANDSCAPED. SIGNS ARE PERMITTED IN SUCH YARDS PROVIDED THEY ARE IN COMPLIANCE WITH THE SIGN ORDINANCE OF THE CITY OF SANTA ANA."

- Parking Count – a summation of the available parking spaces

PARKING COUNT

TOTAL SURFACE PARKING COUNT: 240
DISABLED PARKING: 7
STANDARD PARKING: 233

- Sheet Index Legend - a listing of sheet numbers and descriptions

- Requestor Statement

PREPARED AT THE REQUEST OF COUNTY OF ORANGE CEO/REAL ESTATE ON DECEMBER 6, 2018

- Title Block – a standard OC Public Works title block containing the following info blocks: project name, sheet type (e.g. Title Sheet, Boundary Sheet, etc.), project limits, date of survey, names of field survey party, mapping technician, office reviewer, and field checker, map scale, Work Order Number (assigned by requesting party), Survey File Number (assigned by OC Survey), drawing file name, sheet number (e.g. 1 of 10), and revisions

COUNTY OF ORANGE
OC PUBLIC WORKS/ENGINEERING SERVICES
OC SURVEY/FIELD SERVICES/BOUNDARY ANALYSIS & MAPPING

ALTA /NSPS LAND TITLE SURVEY
2229 S.YALE STREET,SANTA ANA,CA

TITLE SHEET

FROM:N/A
TO:N/A

MARK	DATE	ITEM
2	12/26	UPDATED PTR 12-26-08
1	12/11	UPDATED PTR 12-07-08

REVISIONS

DATE OF SURVEY: NOV. 30, 2018	BY: E. MEDINA PLS & PARTY	SHEET NO.
TOPOGRAPHIC MAPPING	BY: K. AHMED	1
OFFICE CHECK	BY: WADE D. WEAVER PLS	of
FIELD CHECK	BY: MICHAEL A. KUBISTY PLS	3
SCALE:N.T.S	W.O. #EX036Y15	
FILE: YALE_2018_TOPOGRAPHICMAP.DWG		

- North Arrow

- Scale for Sheet Index and Location Map

ALTA/NSPS Boundary Sheet Contents:

- Title Block
- Map Scale
- North Arrow
- Symbol and Line Legend
- Abbreviation Legend
- Mapping Graphics, Depicting:
    1. Boundary Lines
    2. Easement Lines
    3. Centerlines
    4. Measured and Record Line/Curve Data (with bearings oriented north when required)
    5. Monument Symbols
    6. Easement Callouts as Related to Preliminary Title Report (PTR)
    7. Parcel and Adjoiner Information
    8. Additional Details Necessary for Clarity
- Legal Description of Property in Question (PIQ)
- Preliminary Title Report Schedule B Exceptions
- Monument Establishment Notes
- Monument Description Notes
- Record References

ALTA/NSPS Topographic Sheet Contents:

- Title Block
- Map Scale
- North Arrow
- Match Lines (with a scale-appropriate degree of overlap)
- Legend (including a Symbol Pattern Key, a Line Pattern Key, and an Area Pattern Key)
- Street Names
- City Names and/or City Limit Lines
- Assessor’s Parcel Numbers (when parcel lines are shown)
- Residence Address
- Contour Labels (major contours always labeled; minor contours also labeled on projects with minimal vertical change)
- Contour Interval
- Pipe Inlet/Outlet Labels (pipe size and material; invert and rim elevations)
- Obscure Feature Labels (features not defined in legend)
- Flood Zone (limits displayed if necessary)
- Zoning Information

- Building Material, Height, Square Footage (at footprint)
- Dimensions to all Features within 5 Feet of Property Line
- Dimensions from Building Footprint to Property Line
- Underground Utility Location and Depth (if applicable)
- Completed “Table A” Items
- Easements displayed, dimensioned, and denoted as related to Preliminary Title Report (PTR)
- Parking Count (to include disabled stalls)
- Property Line
- Depiction of Adjoiner Lines
- Labeling of Adjoiners
- Centerlines of Facilities and Streets
- Any Other Pertinent Title Information

Example:
An example of a typical OC Survey ALTA/NSPS Land Title Survey can be found here.

Components of a Record of Survey

The technical requirements of a Record of Survey are defined in §§ 8763-8764.5 and § 8771.5 of the Professional Land Surveyors’ Act (Business and Professions Code).
More specific requirements can be found in the Guide to the Preparation of Records of Survey and Corner Records, published by the County Engineers Association of California. In addition, OC Survey - Map Check and ROW Services has developed a Record of Survey check list, which provides a listing of elements required by the County.

Components of a Corner Record

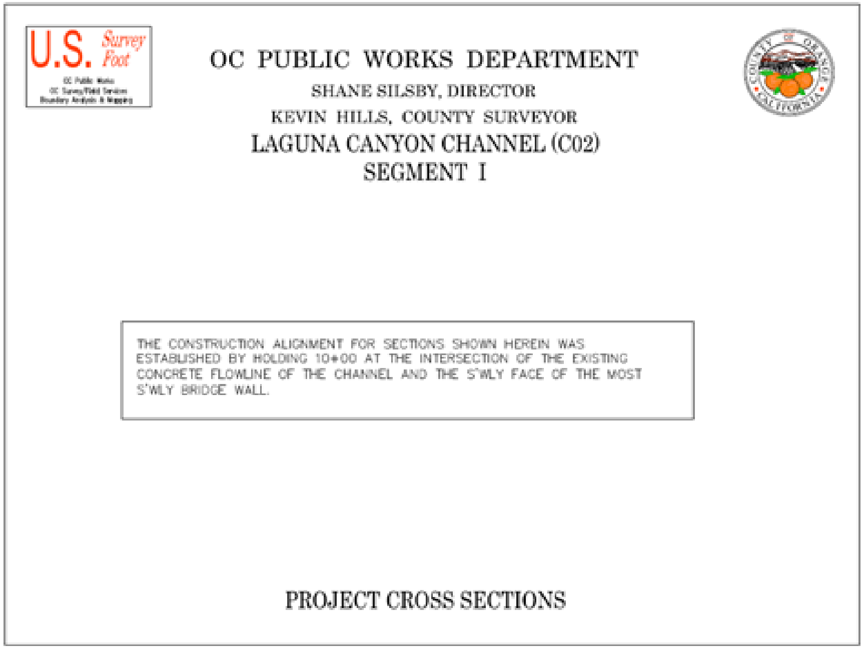
The Corner Record Form is referenced in § 8773.1 of the Professional Land Surveyors’ Act (Business and Professions Code). A copy of the most current Corner Record form can be found in the forms section of the Board for Professional Engineers, Land Surveyors, and Geologists (BPELSG) website. Corner Records prepared by or for OC Survey shall be prepared in CADD and utilize the most current OC Survey Corner Record Template.

Components of Cross Sections

Cross section reports are typically created as a supplement to a topographic or scour study survey, and therefore lack several of the elements included in other mapping products. Required elements are detailed below:

Title Page:

- Below is an example of a typical cross section report title page:



Subsequent Pages:

Each subsequent page of a cross section report represents one specific station along an alignment and shall contain the following elements:

- Title – including street or facility name and the station represented on that specific page
- Statement of Units – a statement defining project units, e.g. US Survey Foot
- Body – a graphical cross sectional representation of one or more surfaces, superimposed upon a grid, with the X axis representing distance left and right of centerline and the Y axis representing elevation
- Line Type Legend – defines the line type for each surface represented within the cross section
- Reference – identifies the record reference used to develop the as-built surface
- Scale – defines the horizontal and vertical scale of cross section data
- Date – date the cross section report was generated

Example:

An example of a typical OC Survey Cross Section Report can be found here.

Components of a Field Note Package

The Components of a Field Note Package are defined in Chapter 13 – Preparation of the Field Note Package.

Mapping Format and Styles

Map Scale:

- Topographic Maps - typically 1″ = 20′ or 1″ = 40′
- Boundary Maps and Records of Survey - range from 1″= 20′ to 1″= 60′ (excluding conceptual views and details)
- Corner Records - typically 1″= 20′, 1″= 30′, or Not to Scale (NTS)
- Field Note Packages - range from 1″ = 20′ to 1″ = 100′ (Monument Location Maps)

Text:

<u>Topographic Maps and the Relevant Portions of ALTA/NSPS Survey Maps:</u>

- Sheet Title - Arial 0.25″
- Legend Titles - Century 0.175″ (Underlined)
- Primary Street and Facility Names - Century 0.25″
- Secondary Street and Facility Names - Century 0.175″
- General - Simplex 0.125″

<u>Boundary Maps, Records of Survey, and the Relevant Portions of ALTA/NSPS Survey Maps:</u>

- Sheet Title - Century 0.20″
- Legend Titles - Century 0.15″ (Underlined)
- Primary Street and Facility Names - Century .25″
- Secondary Street and Facility Names - Century 0.15″
- Adjoiners - Swis721 BlkOul BT 0.15″
- Emphasized Information (e.g. deed PIQ, Basis of Bearings) - Arial 0.15″ (Underlined as necessary)
- General - Simplex 0.10″

<u>Corner Records:</u>

- Legend Titles - Century .125″ (Underlined)
- Street and Facility Names - Century 0.15″
- Adjoiners - Swis721 BlkOul BT 0.10″
- General - Simplex 0.10″

<u>Cross-Sections:</u>

- Title - Times New Roman 0.225″
- Horizontal/Vertical Grid Labels - RomanS 0.10″
- Line Legend and Scale - Calibri 0.15″
- Date Prepared - Italicized Calibri 0.15″

Line Types:

The drawing templates (.dwt) for each type of map will dictate the line types by layer. Examples shown below demonstrate similarities and differences of common line types as they appear on different types of maps.

<u>Boundary Maps and Records of Survey:</u>

- Street Centerline - Center2 (0.216mm)
- Channel Centerline - Center (0.216mm)
- Right of Way Line - Continuous (0.35mm)
- Parcel/Lot Line - Continuous (0.18mm)
- Parcel in Question - Continuous (0.65mm)

<u>Topographic Maps:</u>

- Street Centerline - Center2 (0.216mm)
- Channel Centerline - Center (0.216mm)
- Construction Centerline - Center2 (0.216mm)
- Right of Way - ROW (0.35mm)
- Parcel/Lot Line - Continuous (0.18mm)
- Parcel in Question - PL (0.35mm)

<u>Corner Records:</u>

- Street Centerline - Center2 (0.35mm)
- Tie Line - Dashed2 (0.254mm)
- Right of Way - Continuous (0.216mm)
- Parcel Line - Continuous (0.216mm)

<u>Cross-Sections:</u>

- Horizontal and Vertical Grid - Continuous (0.05mm)
- Center Axis - Center2 (0.40mm)
- Individual Surfaces (as-built, existing, etc) – Dictated by Civil 3D line styles

Sheet Sizes:

- Topographic and Boundary Maps - 24”x36”
- Records of Survey - 18”x26”
- ALTA/NSPS - 24”x36”
- Corner Records - 8.5”x11”
- Cross-Sections - typically 8.5”x11”, but may vary by project

Deliverables

CADD Deliverable:

- Topographic Mapping – clean drawing file (purge, audit, no xrefs) in a zipped folder (utilizing "eTransmit" command) with proper naming convention
- Boundary Mapping – clean drawing file (purge, audit, no xrefs) in a zipped folder (utilizing "eTransmit" command) with proper naming convention
- Record of Survey – clean drawing file (purge, audit, no xrefs) in a zipped folder (utilizing "eTransmit" command) with proper naming convention
- ALTA/NSPS Survey – clean drawing file (purge, audit, no xrefs) in a zipped folder (utilizing "eTransmit" command) with proper naming convention
- Cross Sections – typically no CADD files are delivered

Naming Convention for Zipped Folder - "Project Name_Survey Type_Year_CAD Files.zip" (e.g. Fairview Channel_Topo_2019_CADD Files.zip)
Naming Convention for CADD Files in Zipped Folder - "Project Name_Survey Type_Year_eTransmit.dwg"

A detailed workflow for "cleaning" the drawing file can be found here: [Final Deliverable Workflow]

PDF Deliverable:

- Topographic Map – signed title sheet with topographic mapping sheets
- Boundary Map – signed title sheet with boundary mapping sheets
- Record of Survey – required signatures together with all mapping sheets
- ALTA/NSPS – required signatures together with all mapping sheets
- Cross Sections – supplemental to the topographic mapping sheets

Naming Convention for PDF File: "Project Name_Survey Type_Year_Signed.pdf"

CHAPTER 15
CARE AND MAINTENANCE OF SURVEY EQUIPMENT

Individual Responsibility

Each employee has a responsibility to reasonably care for County equipment. Reasonable care includes appropriate use of all equipment, proper storage and transport, protection from extreme environmental elements, and preventing theft. In the event that equipment is lost, damaged, or stolen, disciplinary action may be taken when damage or loss is a result of operator's negligence or abuse.

Crew Inventory

Each Party Chief is responsible for maintaining an inventory of all equipment assigned to them, including fixed asset ID numbers on major items. This inventory should include instruments (total stations, levels, GNSS receivers, etc.), data collectors, tribrachs, prisms, tripods, GNSS rover rods, layout rods, leveling rods, radio equipment, and any other items beyond basic hand tools. Should an item turn up missing which cannot be located during a cursory search, the first line supervisor should be notified.

Stolen Equipment

In the event County equipment is stolen or damaged, the employee assigned the equipment must follow the following procedures for reporting the loss:

- Notify the first-line supervisor immediately
- Contact the Orange County Sheriff and/or the local law enforcement agency with jurisdiction in the area where the loss occurred; file a stolen property report with said agency
- File an internal report, documenting a complete description of the equipment (make, model, serial number, fixed asset ID number, etc.)

Guidelines for Day-to-Day Care and Use

Following are specific rules ("shall") and general guidelines ("should") for daily care and use of County equipment:

- Before use, instruments should be visually inspected for damage. The instrument exterior should be cleaned frequently in accordance with manufacturer's recommended procedures.
- If an instrument has been exposed to moisture, it should be removed from the case and wiped dry with a clean cloth, and the case left open overnight.

- Instruments shall be stored each night at the field office, in the vault. Under no circumstances are instruments to be left inside a County vehicle overnight.
- Radios and data collectors shall be stored each night within the Party Chief's locked office or in the vault. Under no circumstances are radios or data collectors to be left inside a County vehicle overnight.
- Instruments shall be transported in their designated shock-proof cases, positioned within the vehicle in such a manner as to limit movement. Under no circumstances are instruments to be transported in the open bed of a truck.
- GPS receivers should be removed from the tripod or rover rod when transporting.
- Tripod-mounted instruments should not be carried "over the shoulder".
- Instruments shall not be left unprotected or unattended in unsecured areas. GNSS receivers may only be left unattended when located within a construction site or County facility. Robotic total stations are particularly vulnerable to theft, even with crew members within close proximity. Robotic operation should only take place within a construction site or County facility, or in an area with minimal vehicular and foot traffic.
- Instruments should not be left unattended during high winds, unless at least two tripod legs are secured by sandbags.
- GPS rover rods that are supported by bipods should not be left unattended (out of arm's reach).
- Data collector screens shall only be manipulated by stylus or finger-tip. Tapping the screen with a sharp object, such as the tip of a mechanical pencil, will cause damage to the screen and is not permitted.

Routine Maintenance Schedule

Following is the recommended schedule for routine maintenance of County equipment:

Check and Adjust Rod Bubbles:

Regular: Every two weeks

Additional: Before an RTK survey is conducted which uses a rod/bipod configuration; any time a rod is dropped or the bubble assembly suffers an impact

Notes: Bubbles of "four-legged" fixed height rods shall be adjusted each morning and checked during each occupation during the course of a static GNSS control or boundary survey.

Check Tribrachs - User:
Regular: Every month
Additional: Before any survey requiring a higher degree of accuracy, such as settlement monitoring or use on the SAR Baseline; any time a tribrach is dropped or the bubble/ optical plummet assembly suffers an impact
Notes: Number all tribrachs; keep a record of dates checked/adjusted

Adjust Tribrachs - Dealer:
Regular: No regular servicing required
Additional: When a tribrach is checked in-house and determined to require adjustment of the optical plummet; when a tribrach continually falls out of adjustment

Clean and Inspect Prism Assemblies:
Regular: Every 2 months
Additional: After exposure to extreme dust conditions; any time a prism assembly suffers an impact

Clean Data Collector and Replace Screen Protector:
Regular: Every 2 months
Additional: After exposure to extreme dust conditions; as needed

Calibration of Total Station - User:
Regular: Every month
Additional: At the beginning of a new project; weekly during the conduct of a control or boundary survey; after long periods of storage; after a change of altitude greater than 1500 feet; after a change in operating temperature greater than 20°; after transport on a particularly jarring roadway (4 x 4); any time the total station (inside protective case) is dropped
Notes: This process shall include tilt compensator calibration, HA/VA collimation, trunnion axis tilt (if applicable), and tracker (auto-lock) collimation. Generate an Instrument Collimation Report using the Style Sheet of the same name and keep on file

Calibration of Total Station - Dealer:
Regular: Every 12 months
Additional: Any time the total station (not inside protective case) is dropped

Calibration of Level (peg test) - User:
Regular: Every month
Additional: At the beginning of a new project; daily during the conduct of a second order precise leveling project; after long periods of storage; after transport on a particularly jarring roadway (4 x 4); any time the level (inside protective case) is dropped
Notes: See Chapter 4 – Differential Leveling for further information on peg test procedures

Calibration of Level - Dealer:
Regular: None
Additional: Any time the level (not inside protective case) is dropped