Chapter 3
Topographic Survey Control

3-1. General

USACE control work can generally be separated into primary project control and secondary project control. Primary control, usually on the State Plane Coordinate System (SPCS) projection, is intended to accommodate all project-related surveying tasks identified for the life of a particular project. Secondary control is intended for more specific applications or densification such as topographic surveying or construction stakeout. The specified USACE control accuracies are listed by activity type in Table 2-1. To explain check surveys designed to establish map accuracies refer to the ASPRS Accuracy Specification for Large-Scale Line Maps included in EM 1110-1-1000.

a. USACE primary project control is sometimes tied into, but rarely adjusted as part of, the National Geodetic Reference System (NGRS). This tie is generally performed by converting geodetic control coordinates into the SPCS or UTM. The grid coordinates of the control monuments are inversed for grid azimuth and grid distance. The azimuth is used as a starting azimuth (or check azimuth) and the ground scale distance is used to check the monuments.

b. The Global Positioning System (GPS) has significantly increased the production of surveys performed by surveyors. The USACE may occasionally require a geodetic survey because of unique construction requirements or project size, but most projects are surveyed based on SPCS or UTM horizontal grids. Independent survey datums or reference systems should not be used for primary project control unless required by local code, statute, or practice. This includes local tangent planes, state High Accuracy Regional Networks (HARN), and un-referenced construction baseline station-offset control. The horizontal datum for the primary control should be the North American Datum of 1983 (NAD 83/86). New projects should be based on the NAD 83 system and some older projects may remain based on the NAD 27 system.

3-2. USACE Control Survey Accuracy Standards

a. Horizontal control standards. The USACE uses the relative point closure survey accuracy standard. The compass rule adjustment procedure will produce this result by dividing the closure by the sum of the traverse lengths. Table 3-1 displays the classification for closure standards.

Table 3-1
USACE Point Closure Standards for Horizontal Control Surveys

USACE Classification	Point Closure Standard (Ratio)
Second-Order Class I	1:50,000
Second-Order Class II	1:20,000
Third-Order Class I	1:10,000
Third-Order Class II	1: 5,000
Fourth-Order - Construction Layout	1:2,500 to 1:20,000

(1) Horizontal point closure. The horizontal point closure is determined by dividing the linear distance misclosure of the survey into the overall circuit length of a traverse, loop, or network line/circuit. When independent directions or angles are observed, as on a conventional survey (i.e., traverse, trilateration, or triangulation), these angular misclosures may optionally be distributed before assessing positional misclosure. The horizontal point closure is also used in USACE as an accuracy measure. In cases where GPS vectors are measured in geocentric coordinates, then the three-dimensional positional misclosure is assessed.

(2) Approximate surveying. Approximate surveying work should be classified based on the survey's estimated or observed positional errors. This would include absolute GPS and some differential GPS techniques with positional accuracies ranging from 10 to 150 feet (2DRMS). There is no order classification for such approximate work.

(3) Higher order surveys. Requirements for relative accuracies or closures exceeding 1:50,000 are rare for most USACE applications. See Table 2-1 for recommended control survey accuracies based on the functional project application. Surveys requiring accuracies of First-Order (1:100,000) or better should be performed using Federal Geodetic Control Subcommittee (FGCS) standards and specifications.

b. Vertical control standards. The vertical accuracy of a survey is determined by the elevation misclosure within a level section or level loop. For conventional differential or trigonometric leveling, section or loop misclosures (in feet) shall not exceed the limits shown in Table 3-2, where the line or circuit length (M) is measured in miles. Procedural specifications or restrictions pertaining to vertical control surveying methods or equipment should not be over-restrictive.

Table 3-2
USACE Point Closure Standards for Vertical Control Surveys

USACE Classification	Point Closure Standard (Feet)
Second-Order Class I	$0.025*M^{0.5}$
Second-Order Class II	$0.035*M^{0.5}$
Third-Order	$0.050*M^{0.5}$
Fourth-Order-Construction Layout	$0.100*M^{0.5}$

NOTE: $M^{0.5}$ is the square root of distance M in miles.

3-3. Reconnaissance and Planning Phase

The reconnaissance phase could be the most important phase of the survey. At this phase of the project the required control accuracy is known. Topographic maps, aerial photographs, tax maps, and basically any mapping information are collected for the area. These maps along with required site visits shall be used to extract slopes, soil characteristics, ground cover, drainage structures, nearby utilities, and other physical evidence. Weather information and rainfall data should be consulted at this time. Additionally, wetlands, historical artifacts, and other like sites shall be identified. All the above information should be consolidated into a reconnaissance report. Photographs are encouraged in this report. Ownership information (name and address) and legal descriptions of privately owned lands should be obtained during this phase. Right-of-entry should be secured on lands not owned or controlled by the Government. Permits may be required and should be obtained at this time.

a. Vertical information is important for horizontal control computations as well as the project vertical control, in general. U.S. Geological Survey (USGS) quadrangles and publications of National Ocean Service (NOS) horizontal/vertical control are invaluable to the beginning of the project. Elevations obtained from quad sheets are often used to estimate the average project height above a datum in order to compute the sea level or ellipsoid reduction factor.

b. The size and type of the project, target scale, and contour interval will define the type and accuracy requirements of the control to be established. Once this has been established, instruments and the measurement system are selected. The measurement systems are:

Triangulation
Trilateration
Traverse
Inertial Surveying
GPS Surveying
Geodetic Leveling
Photogrammetry/Analytical Control

Of the above measurement systems, USACE typically uses traverse or GPS for horizontal control.

c. After the information has been sorted and permission to enter the property has been granted, the survey party will go to the field and recover the monuments which will be of benefit to the project. At the same time traverse stations are strategically set and lines may be cleared under the conditions of the right-of-entry. Monuments set by different agencies should be identified and noted. Information supporting the unfamiliar monuments may be obtained from the respective agencies. Without this supporting information, the positions of these monuments may never be of value to the project. Unanticipated monuments that are found should be noted, because the monument may have value that was not apparent when the party first walked the project. The key is to locate anything that remotely resembles a monument in the field, but don't hold this monument as equal weight with other known monuments in the traverse adjustments unless the history is known. Use the coordinate as a check unless no doubt exists about the position.

d. Plane coordinate systems can be used for large projects. The horizontal control is installed, measured, and adjusted by conventional traverse methods. This traverse is tied to a geodetic control network by including a geodetic monument (or monuments) in the traverse or by locating the geodetic monuments by ties (spur lines).

3-4. Primary Survey Control

Primary project control is set to establish survey control over large areas. USACE primary project control may be geodetic or SPCS. Geodetic primary project control may be obtained from the National Geodetic Survey (NGS). A vast amount of high order survey information is compiled and adjusted by this organization to provide horizontal control and benchmarks in the United States. The latest adjustment of these data, including Canadian and Mexican geodetic survey data, establishes the vertical and horizontal datums for the North American continent. The newest vertical datum is the North American Vertical Datum 1988 (NAVD 88). The latest horizontal datum is the NAD 83. Connections with the NGRS shall be subordinate to the requirements for local project control.

3-5. GPS Survey Control

New projects may exist in areas that have no primary control available. GPS surveying has proven highly effective in transferring control from monuments outside the project. Differential GPS applications are capable of establishing control to accuracies better than 1:100,000. Although GPS is referenced to World Geodetic System of 1984 (WGS 84) rather than Geodetic Reference System of 1980 (GRS 80, ellipsoid definition used in NAD 83), the difference in resulting coordinates obtained through differential applications is negligible. Therefore, for differential applications, GPS can be assumed to be a NAD 83-referenced system. For details of GPS operation refer to EM 1110-1-1003.

3-6. Secondary Control for Topographic Surveys

Secondary control traverses are constructed as closed loop traverses, closed traverses, or nets, on flat plane coordinate systems. Loops can be adjusted by compass rule or least squares. Nets are best adjusted by least squares. Nets evolve in projects from a closed loop traverse where cross-tie traverses are later tied across the project. Reductions are made to survey data to project the control to the reference vertical datum. Another reduction for state plane coordinates places the control on the flat plane projection. Two points on the surface of the earth are really separated by an arc distance. If the earth radius varies in length then so varies the distance between the same two points in a direct relationship. Following these two reductions, any distance between the control points will appear to be in error until the distance is divided by the combination of the two scale factors, called the combined scale factor. A local secondary system avoids this multiplication/division exercise to allow the surveyor to make environmental corrections to measurements without additional conversions. Later the entire project will be converted from local to datum plane after field operations have ceased. Horizontal control for topographic surveys and construction surveys is designed primarily to provide positions within allowable limits. With the electronic equipment used today, the largest type of error is the blunder. Systematic error can be reduced through calibration checks. Random error is more significant for primary survey control and stakeouts of large expensive structures. For topographic control, angles should be doubled and distances should be recorded ahead (AHD) and back (BK) at each traverse station. Thus, each measurement is recorded twice. This procedure is only recommended, not required, to avoid blunders. The traverse is adjusted, and a relative error of closure is obtained. This relative position closure provides a statement of protection against blunders and possibly some systematic errors. The error of closure of this control should be slightly less than the primary control used to set up this third- or fourth-order traverse. As long as the points located from this control can meet the requirements (including points far from the control but still within the map sheet), then the control is acceptable. This control is considered to be independent of the control used to check the topographic map.

3-7. Plane Coordinate Systems

If the project is 6 miles or less, then a flat plane can represent all the control points, although all USACE projects should be on a SPCS or a UTM system. The only exception is a project datum (local plane) system used for construction.

a. Each state in the United States has state plane grids which stretch 158 miles or less in a flat plane. Two types of map projections are typically used to project the curved earth surface onto a flat plane. States elongated in the north-south direction typically adopt a Transverse Mercator System. Eighteen states use this system exclusively. States which are elongated in an east-west direction use the Lambert System. Thirty-one states use the Lambert System. The states of Alaska, Florida, and New York use both systems. Also, Alaska uses an oblique Mercator projection on the southeast panhandle.

b. Another plane coordinate system used by the military or on large civil projects is the UTM coordinate system. The UTM system is divided into sixty (60) longitudinal zones. Each zone is six (6) degrees in width extending three (3) degrees on each side of the central meridian. The UTM system is applicable between latitudes 84° N to 80° S.

c. For first- or second-order surveys, all distances will be reduced by the first two of the following corrections:

(1) Reduction to reference elevation datum.

(2) Scale factor.

(3) Curvature correction.

d. The horizontal curvature correction is used for precise surveys where distances are measured in excess of 12 km (7.5 miles). The arc/chord distance change is less than 1 part per million (ppm) and is not significant for most USACE applications. NOTE: The curvature correction, if necessary, is applied to field-measured distances before any other reductions or adjustments are performed. The combined scale factor will be applied to the rectangular coordinates if only one point is held fixed for the traverse adjustment.

e. The vertical curvature correction may be significant for precise surveys using electronic equipment. Optical levels used for ordinary survey work should not be affected by curvature, since a balanced shot to either the backsight or the foresight should not exceed 300 feet.

f. The first two corrections listed above, reference elevation reduction factor and scale factor, are often combined in practice. The multiplication of these factors is termed the "combined factor." Survey parties using primary control for construction stakeout purposes need the combined scale factor listed on the particular construction drawing sheet being used for the stakeout. The survey party divides inversed grid control coordinates by this correction to check measurements between the recovered traverse stations. If the field measurements check, the stakeout proceeds.

g. If any of these corrections are applied to survey measurements, a formal note must be printed on all drawings depicting the horizontal control. This note shall be the link between the points on the ground surface and the plane coordinate system used for the project.

3-8. Scale Factor Considerations

USACE projects typically use SPCS projections, therefore coordinate problems may be present in high elevation locations or route surveys extending for long distances.

a. Projects in high elevations. For projects in high elevations mapped to large scale, a project datum may be necessary to separate the survey measurements at ground scale from coordinates reduced down to the SPCS projection. This project datum eliminates blunders. The term "project datum" is used in lieu of the term "local datum" to mean the project datum is not translated or rotated from the SPCS orientation. Local datums in USACE can be rotated to station numbers and offsets from center lines.

b. Constructing a project datum. To construct a project datum, a digital computer file of SPCS coordinates is typically used. A large constant (number) is subtracted from every northing and easting to identify the coordinates as entirely different from the SPCS coordinates. Next all northing and eastings are divided by the selected combined scale factor to put the coordinates at the ground scale. The project datum is now established.

c. Project datum example. For a project datum example, Figure 3-1 can be used to illustrate the procedure. The traverse stations labeled A1, A2, A3,... C1, C2, C3 may be used to establish the project secondary control. The digital computer file of project secondary coordinates is used to establish a project datum for field use. Say 20,000,000 was subtracted from all the northings and eastings to alert field crews that this coordinate file is not on the SPCS. Next the combined scale factor of 0.994632 from Figure 3-1 is divided into all northings and eastings. Now field crews can operate in a project datum without confusion about control distances that don't match ground distances. A note shall be added relating the project datum back to the SPCS. The coordinates shall be titled "Project Datum Coordinates." Figure 3-2 represents a worksheet for use in converting state plane coordinates to project datum coordinates.

d. Reduction to sea level. Distances between geodetic monuments are reduced to sea level for the North American Datum of 1927 (NAD 27). Sea level was the intended datum for North America, modeled from the Clark 1866 spheroid. The vertical datum used relative to 26 primary tide station local sea levels is the National Geodetic Vertical Datum of 1929 (NGVD 29). Distances between geodetic monuments of NAD 83 are reduced to the ellipsoid. The ratio used to compute this scale factor is

$$\textit{Base length / ground distance} = R / (R + H + N)$$

where

R is the mean earth radius = 20,906,000 feet

Example: A distance of 1,000.00 feet is measured between monuments where

Mean height (H) = 2,416.1 feet

Mean geoid height (N) = -58.4 feet

Ellipsoid factor

$$= R / (R + H + N)$$

$$= 20{,}906{,}000/(20{,}906{,}000 + 2{,}416.1 + -58.4)$$

$$= 20{,}906{,}000/20{,}908{,}357.7 = 0.99988724$$

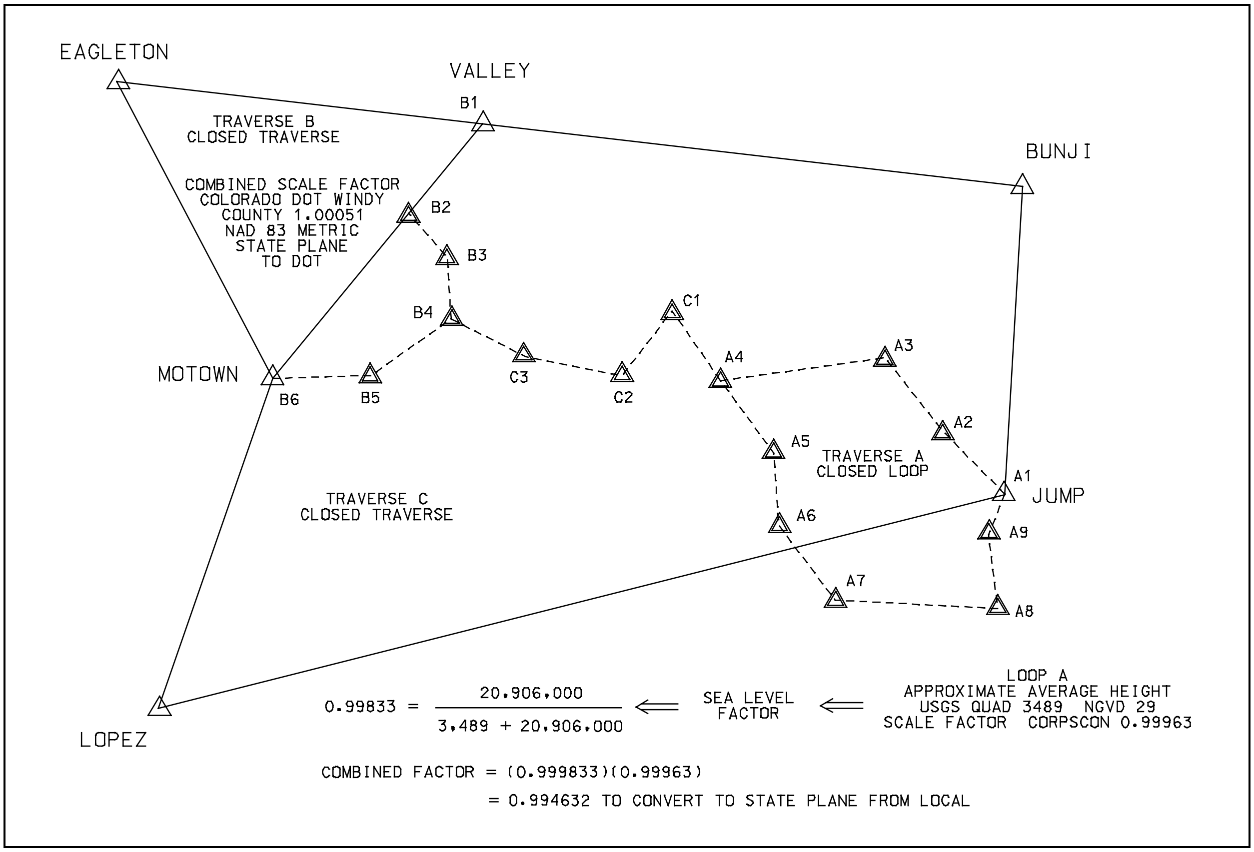

Figure 3-1. Primary, secondary project control

Therefore

*Base length = ground distance * ellipsoid factor*

= 1000 feet * 0.99988724

= 999.89 feet

e. Scale is the grid distance divided by the geodetic distance.

Scale factor = grid distance / ground distance

The grid distance is the *scale factor* times the geodetic distance. Scale factors may be less than, equal to, or greater than 1.0. Lambert Conic projections vary scale in the north-south direction. UTM and Transverse Mercator (TM) projections vary in the east-west direction.

3-9. Control Checks

If the existing control is geodetic, the lengths between the monuments will have been reduced to reference vertical datum. The azimuth between them will be a geodetic azimuth and any reference monuments in the vicinity will have geodetic azimuths. Field checks will require corrections to the measurements in many cases. For high elevations, distances could measure longer by a tenth or more in a thousand feet.

a. Coordinate transformations. Differential GPS units can measure the distance between two points based on the datum selected in the GPS receiver unit. A three-dimensional vector is established between the master station and the remote. The initial computation in both receivers is based on the WGS-84 ellipsoid and the Cartesian vector relationship from the earth-center to each unit. Vector subtraction between the earth-center vectors provides the WGS 84 Cartesian vector between the units. Any errors are assumed to be subtracted out during this process. Further coordinate transformations provide the geographic coordinate based on the map projection parameters.

Work Sheet for Converting State Plane Coordinates to Project Datum									
Project Name		Zone (North or South		Minimum Latitude	Maximum Latitude	Mean Latitude	Mean Elevation		Mean Geoid Height
(S.F.) Obtained by using the mean latitude from the table k value		(E.F.) $E.F. = \frac{R}{R+N+H}$ R = assumed radius of earth of 6,372,000 (meters) H = mean elevation from sea level for project (meters) N = mean geoid height for project (meters)			(C.F.) $S.F. \times E.F. = C.F$		(N) and (E) Project Datum $N = \frac{Y}{C.F.} + 1,000$ $E = \frac{X}{C.F.} + 1,000$		Conversion to US Survey $N \times \frac{39.37}{12}$ Foot $E \times \frac{39.37}{12}$
Point ID	(Y) State plane Northing (meters)	(X) State Plane Easting (meters)	(S.F.) Mean Scale Factor (8 significant figures)	(E.F.) Mean Elevation Factor (8 significant figures)	(C.F.) Combined Factor (8 significant figures)	(N) Project Datum Northing (meters)	(E) Project Datum Easting (meters)	Project Datum Northing (feet)	Project Datum Easting (feet)
Calculated By:					Checked By:				

Figure 3-2. Work sheet for converting state plane coordinates to project datum

b. Vertical component. The vertical component of a GPS position is usually converted to a vector component in the normal direction to the surface at the receiver. The surface is the ellipsoid, not the geoid. At any point, the ellipsoid will generally be different than the geoid (mean sea level with undulations) or the elevation where the measurements are taken.

c. Closed loop traverse. The first step in checking a closed traverse is the addition of all angles. Interior angles are added and compared to $(n-2)*180^o$. Exterior angles are added and compared to $(n+2)*180^o$. Deflection angle traverses are algebraically added and compared to 360^o. The allowable misclosure depends on the instrument, the number of traverse stations, and the intention for the control survey.

$$c = K * n^{0.5}$$

where

c = allowable misclosure

K = fraction of the least count of the instrument, dependent on the number of repetitions and accuracy desired (typically 30" for third-order and 60" for fourth-order)

n = number of angles

Exceeding this value, given the parameters, *may* indicate some other errors are present, of angular type, in addition to the random error. The angular error is distributed in a manner suited to the party chief before adjustment of latitude and departures. Adjustment of latitudes and departures is the accepted method in the USACE. If GPS is used, latitude, departures, and adjusted line lengths are computed after adjustment to obtain the error of closure and the relative point closure. The relative point closure is obtained by dividing the error of closure (E_C) by the line lengths.

$$\text{Relative point closure} = E_C / \Sigma \text{ of the distances}$$

d. Closed traverse between two known control points. In order to establish a solid field technique, the initial azimuth shall be checked in the field with GPS or astronomic observations before this type of traverse is continued. The extra time in minutes will save many work-hours of re-computation if the beginning or ending azimuth is not in the same meridian alignment.

(1) The surveyor may decide to hold both ends of the traverse as fixed and only adjust the measurements in between. The misclosure between the two fixed points provides the expected error at the distance from the first control point. The surveyor holds one control point fixed and proportionally spreads (prorates) error between the points into the cross traverse as a fraction of the distance between the control points. The assumed error of the misclosure between the control points must be within the allowable limits of error for this procedure. For example, if control is in state plane coordinates (Third-Order, Class I) then the closure must be at least 1:10,000. If a traverse has legs which add up to 8764.89 feet then

$$\text{8,765-foot length} * (\text{1-foot } E_C) / (\text{10,000-foot length}) = 0.88 \text{ foot of allowable } E_C$$

(2) The procedure for adjusting this type of traverse begins with angular error just as in a loop traverse. To determine the angular error a formula is used to generalize the conversion of angles into azimuth. The formula takes out the reciprocal azimuth used in the backsight as (n-1) stations used the back-azimuth as a backsight in recording the angles.

$$A_1 + \alpha_1 + \alpha_2 + \alpha_3 + ... + \alpha_n - (n-1)(180^o) = A_2$$

(3) If the misclosure is exceeded, the angular error may have been exceeded or the beginning and ending azimuths are in error or oriented in different meridian alignments.

(4) GPS points can aid the closure process by establishing point pairs at the endpoints with conventional surveys between the point pairs. Do not exceed 15 km in distance from the master station to the point pairs. Also, separate the point pairs by a distance of at least 400 m to make the constant GPS error (cm ± ppm) insignificant. If GPS is not available, sun shots can be taken at both ends of the traverse. The sun shot will only establish an azimuth reference where GPS provides azimuth and position with point pairs.

(5) If beginning and ending azimuths were taken from two traverses, and the angle repetitions were found to be at least an order of magnitude better than the tabulated angular error, the ending azimuths may contain a constant error which may be removed to improve the allowable error. GPS or astronomic observations may be used to find the discrepancy if the benefit of this

procedure is worth the extra cost of the improved traverse to the total project costs.

(6) The beginning and ending azimuths are measured and converted to grid. The check grid azimuth may then be compared to the grid azimuth of the traverse (if the traverse was in grid).

$$\varepsilon = \textit{measured} - \textit{true}$$

$$\Delta\alpha = \textit{recorded azimuth} - \textit{GPS azimuth}$$

$$\Delta\alpha_{BK} = \Delta\alpha_{AHD}$$

$$\Delta\alpha_{BK} - \Delta\alpha_{AHD} = 0$$

$$\textit{true} = \textit{measured} - \varepsilon$$

(7) The error is algebraically subtracted from the angle misclosure. The absolute value of the misclosure should be less than the initial misclosure because of external plus internal error. This new misclosure is checked against the allowable error. If the misclosure is now acceptable, proceed with adjustment. The angular misclosure is spread as a cumulative azimuth error from the beginning or ending station. For example, if the angular misclosure was 2 minutes in excess of the above formula, and nine angles were recorded, then

*2 minutes * (60 seconds / 1 minute) * (1/9) angles*
= 13.33 seconds per station

Subtract (13.3)(1) = 13 seconds from azimuth 1

Subtract (13.3)(2) = 27 seconds from azimuth 2

Subtract (13.3)(3) = 40 seconds from azimuth 3
.
.
.
Subtract (13.3)(9) = 120 seconds = 2 minutes from azimuth 9

Note: The significance of seconds depends on the least count of the instrument.