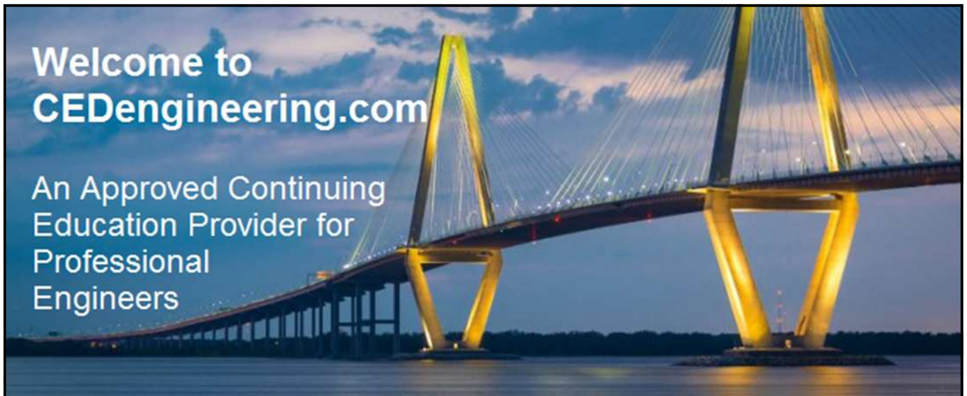

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CONTROL AND TOPOGRAPHIC SURVEYING

Course No: L03-305 - Course Credit: 3 PDH

Prepared by:

Gilbert Gedeon, P.E.

Hosted by:

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LEARNING OBJECTIVES

1. To understand vertical control survey techniques, including second-order and third-order leveling methods, and their role in establishing accurate elevation data.
2. To explore the procedures for calibration, adjustment, and maintenance of leveling instruments, rods, and survey accessories to ensure reliable measurements.
3. To learn about accuracy standards applied in engineering, construction, and topographic surveys, including specifications for mapping, geospatial products, and control surveys.
4. To examine the different categories of survey accuracy standards, such as geodetic control, photogrammetric mapping, cadastral surveys, and hydrographic surveys.
5. To familiarize with geodetic reference systems, including geodetic coordinates, datums, and the WGS 84 reference ellipsoid.
6. To understand horizontal coordinate systems, including geographic coordinates, State Plane Coordinate Systems, and the Universal Transverse Mercator (UTM) system.
7. To gain insights into key concepts such as scale factors, grid convergence, and elevation relationships within coordinate systems.
8. To learn about datum transformations and conversions, including the transition between different reference systems such as NAD 27 and NAD 83.

PRIMARY CONTROL SURVEYS FOR PROJECT MAPPING

SECTION II. Vertical Control Survey Techniques

General

- Vertical control surveys provide a basic framework for controlling elevations on facility mapping projects.
- The purpose of vertical control surveys is to establish elevations on rigid benchmarks throughout the project area.
- These benchmarks can then serve as points of departure and closure for leveling operations and as reference benchmarks during subsequent construction work.
- The NGS, USGS, other Federal agencies, and many USACE commands have established vertical control throughout the CONUS.
- Unless otherwise directed, these benchmarks should be used as a basis for all vertical control surveys.
- Descriptions of benchmark data and their published elevation values can be found in data holdings issued by the agency maintaining the project/installation.
- Information on USACE maintained points can be found at District or Division offices.
- This section focuses on Second-Order and Third-Order vertical control techniques performed using differential leveling instruments.

Figure 1: Sokkia B20 automatic level.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- a. Differential leveling.
  - With differential leveling, differences in elevation are measured with respect to a horizontal line of sight established by the leveling instrument.
  - Once the instrument is leveled (using either a spirit bubble or automated compensator), its line of sight lies in a horizontal plane.
  - Leveling comprises a determination of the difference in height between a known elevation and the instrument and the difference in height from the instrument to an unknown point by measuring the vertical distance with a precise or semi-precise level and leveling rods.
  - Digital (or Bar Code) levels are used to automatically measure, store, and compute heights, and are capable of achieving Second-Order or higher accuracies.
  - When leveling in remote areas where the density of basic vertical control is scarce, the semi-precise rod is generally used.
  - The semi-precise rod should be graduated on the face to centimeters and the back to half-foot intervals.

- When leveling in urban areas or areas with a high density of vertical control where ties to higher-order control are readily available, the standard leveling rods are used--e.g., a Philadelphia rod graduated to hundredths of a foot.
- Other rods that are graduated to centimeters can be used.
- Both types of rods are furnished with targets and verniers that will permit reading of the scale to millimeters or thousandths of a foot if required by specifications.
- This is generally not required on lower-order level lines.
- Standard stadia rods may also be used for lower-order level lines.
- The stadia rod is graduated to the nearest 0.05 ft, or two centimeters.
- These rods are generally equipped with targets or verniers, but if project specifications require, they can be estimated to hundredths of a foot.

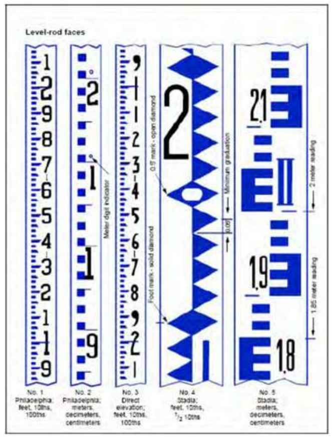

Figure 2: Traditional rectangular cross-section leveling rods showing a variety of graduation markings.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- b. Trigonometric leveling.
  - This method applies the fundamentals of trigonometry to determine the differences in elevation between two points by observing a horizontal distance and the vertical angles above or below a horizontal plane.
  - Trigonometric leveling is generally used for lower-order accuracy vertical positioning; however, it is sufficiently accurate for radial topography when elevations of features are cut in by a total station.
  - Trigonometric leveling is especially effective in establishing control for profile lines, for strip photography, and in areas where the landscape is steep.
  - With trigonometric leveling operations, it is necessary to measure the height of instrument (HI) and rod target above the monuments, the slope distance (s), the vertical angle (a), and the rod intercept.
  - From this data, the vertical difference in elevation (DE) can be computed using the sine of the vertical angle and applying the rod difference.
  - Refinements to this technique include doubling vertical angles, taking differences from both stations, and using the mean values.
  - If the horizontal distance is known between the instrument and the rod, it is not necessary to determine the slope distance.
  - The instrument most commonly used for trigonometric leveling is a directional theodolite or Total Station.
  - Manufacturer specifications and procedures should be followed to achieve the desired point closure standards.

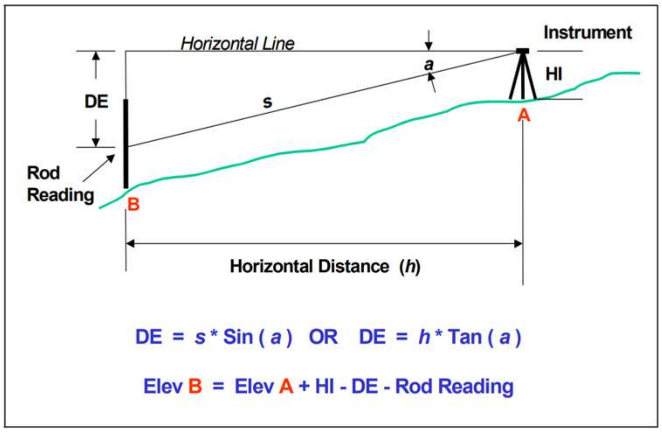

Figure 3: Trigonometric leveling.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

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PRIMARY CONTROL SURVEYS FOR PROJECT MAPPING (CONT'D)
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- (1) Recording vertical trigonometric observations (zenith distances--ZD) is the same for all orders of accuracy.
  - Vertical observations may be recorded in a standard field book, on DA Form 5817-R (Zenith Distance/Vertical Angle), on an equivalent single-sheet recording form, or a data collector.
  - In all cases, complete documentation will be performed in the field.
  - The following information is typically required:
    - The HI above the station (recorded to the nearest 0.01 meter or foot).
    - A sketch of the observed target (that shows the point observed on the target) at the bottom of the object-observed column.
    - The height of the observed rod reading (height of target--HT) above the station being observed (recorded to the nearest 0.01 meter or foot).
    - A sketch showing any target's adjoining stations.
      - ✓ This sketch should be drawn in the bottom of the remarks column.
      - ✓ All possible points that may be observed should be measured and recorded to the nearest 0.01 meter or foot.

- (2) During vertical observations, the time of the first observation of the first position and the time of the last observation of the last position are recorded.
  - The times are recorded to the nearest whole minute.
- (3) Vertical observations may alternatively be abstracted onto DA Form 1943 (Abstract of Zenith Distances) at the station site by the observing party.
  - Vertical observations recorded as vertical angles are converted to ZDs before abstracting.
  - Targets or signals shown to other stations are sketched and dimensioned at the bottom of the form.
  - If a target or signal is changed during the day, the time of the change and the new dimensions are also entered.

- c. Trigonometric elevations over longer lines.
  - Trigonometric elevations over longer lines may need to be corrected for curvature and refraction.
  - These corrections are insignificant (< 0.02 ft) and unnecessary for topographic survey distances of 1,000 feet or less.
  - The type of correction used depends on whether the long line was occupied at each end.
  - The following formula (Wolf and Brinker 1994) is used to determine the combined curvature and refraction correction for trigonometric elevations observed over longer lines.

$$h\ (feet) = 0.0206\ (F)^2 \qquad (1)$$

  - where
    - h = combined correction for curvature and refraction in feet
    - F = length of observed line in thousands of feet

- As an example, given a 2,000 ft line, the combined correction would be 0.0206 (2) 2 = 0.08 ft, or about 0.1 ft.
- For longer lines, these approximate computations are not accurate; however, there are few applications requiring trigonometric leveling over longer lines given GPS methods will yield more accurate results.
- If long-line trigonometric leveling is required, consult the NOAA/NGS for more accurate observing procedures and computations.

- d. Barometric leveling.
  - This method uses the differences in atmospheric pressure as observed with a barometer or altimeter to determine the differences in elevation between points.
  - This method is the least accurate of determining elevations.
  - Because of the lower achievable accuracies, this method should only be used when other methods are not feasible or would involve great expense.
  - Generally, this method is used for elevations when the map scale is to be 1:250,000 or smaller.

- e. Reciprocal leveling (Valley or River Crossings).
  - Reciprocal leveling is a method of carrying a level circuit across an area over which it is impossible to run regular differential levels with balanced sights.
  - Most level operations require a line of sight to be less than 300 or 400 feet long.
  - However, it may be necessary to shoot 500-1,000 feet, or even further, in order to span across a river, canyon, or other obstacle.
  - Where such spans must be traversed, reciprocal leveling is appropriate.
  - The reciprocal leveling procedure can be described as follows.
  - Assume points "A" and "B" are turns on opposite sides of the obstacle to be spanned where points A and B are intervisible.
  - Two calibrated rods are used, one at point A, and the other at point B.
  - With the instrument near A, read rod at A, then turn to B and have target set as close as possible and determine the difference in elevation.
  - Leaving rods at A and B, move the instrument around to point B, read B, then turn to read A and again determine the difference in elevation.
  - The mean of the two results is the final height difference to be applied to the elevation of A to get an elevation value for point B.

- If the long sight is difficult to determine, it is suggested that a target be used and the observations repeated several times to determine an average value.
- For more precise results it will be necessary to take several foresights, depending on the length of the sight.
- It is typical to take as many as 20 to 30 sightings.
- When taking this many sightings, it is critical to relevel the instrument and reset the target after each observation.
- Reciprocal leveling assumes the conditions during the survey do not change significantly for the two positions of the level.
- Reciprocal leveling with two instruments should never be done unless both instruments are used on both sides of the obstacle and the mean result of both sets used.
- The use of two instruments is advised if it is a long trip around the obstacle.
- Reciprocal leveling is effective only if the instruments used will yield measurements of similar precision.

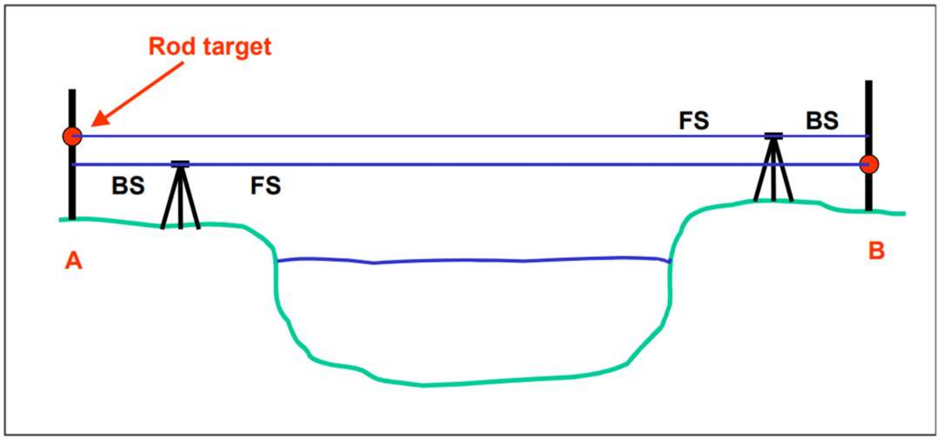

Figure 4: Trigonometric leveling.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- f. Two rod leveling.
  - In order to increase the productivity in precise leveling operations, it is advisable to use two rods.
  - When the observations are completed at any instrument setup, the rods and the instruments are moved forward simultaneously.
  - An even number of setups should be used to minimize the possible effects of rod index error.
  - Two rods are recommended when using an automatic level, as this takes full advantage of the productivity possible with this type of instrument.

Second-Order Leveling

- a. General.
  - As shown in the figure on the following slide, a leveling operation consists of holding a rod vertically on a point of known elevation.
  - A level reading is then made through the telescope to the rod, known as a backsight (BS), which gives the vertical distance from the ground elevation to the line of sight.
  - By adding this backsight reading to the known elevation, the line of sight elevation, called "height of instrument" (HI), is determined.
  - Another rod is place on a point of unknown elevation, and a foresight (FS) reading is taken.
  - By subtracting the FS reading from the height of instrument, the elevation of the new point is established.
  - After the foresight is completed, the rod remains on that point and the instrument and back rod are moved to forward positions.
  - The instrument is set up approximately midway between the old and new rod positions. The new sighting on the back rod is a backsight for a new HI, and the sighting on the front rod is a FS for a new elevation.
  - The points on which the rods are held for foresights and backsights are called "turning points."
  - Other foresights made to points not along the main line are known as "sideshots."
  - This procedure is used as many times as necessary to transfer a point of known elevation to another distant point of unknown elevation.

STA	BS (+)	HI		FS (-)	ELEV	DESCRIPTION
BM Hood-2	6.35	630.22			623.87	Bronze disk ...
TP 1				1.96	628.26	
TP 2						
TP 3						
TP 4						

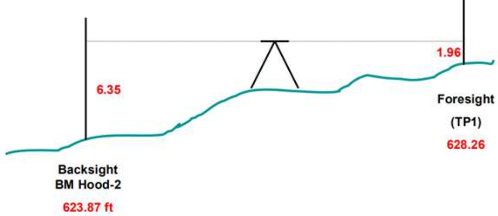

Figure 5: Differential leveling--example of one setup between benchmark and TP 1.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- b. Leveling accuracy.
  - Second-Order leveling consists of lines run in only one direction, and between benchmarks previously established by First-Order methods.
  - If not checking into another line, the return for Second-Order--Class I level work should check within the limits of 0.025 times the square root of M feet (where "M" is the length of the level line in miles), while for Second-Order Class II work, it should check within the limits of 0.035 times the square root of M feet.
  - Ties to two or more benchmarks are always recommended in order to verify stability of the fixed benchmarks.

- c. Leveling equipment.
  - The type of equipment needed is dependent on the accuracy requirements.
  - Examples of precise leveling instruments are shown in the figure below.

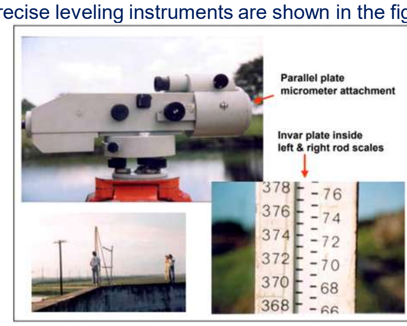

Figure 6: Zeiss Ni1 automatic level with parallel plate micrometer attached--precise double-scale Invar rod with constant 3.01550-meter difference in left and right scales.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- (1) Second-Order leveling instrument.
  - Second-Order leveling instruments require a relatively precise level.
  - Often a graduated parallel plate micrometer is built into the instrument to allow reading to the nearest 0.001 of a unit.
  - The sensitivity of the level vial, telescopic power, focusing distance, and size of the objective lens are factors in determining the precision of the instrument.
  - Instruments are rated and tested according to their ability to maintain the specified order of accuracy.
  - Only those rated as precise geodetic quality instruments may be used for Second-Order work.

- (2) Precise level rods.
  - Precise level rods are normally used when running Second-Order levels.
  - Both traditional rods and bar code type rods may be used.
  - The rods may be of one piece, invar strip type, with the least graduation on the invar strip of 1 centimeter.
  - The front of the rod is graduated in meters, decimeters and centimeters on the invar strip.
  - The back of the rod is graduated in feet and tenths of feet, or yards and tenths of yards.
  - Rods with similar characteristics are paired and marked.
  - The pairings must be maintained throughout a line of levels.
  - The invar strips should be checked periodically against a standard to determine any changes that may affect their accuracy.
  - The precise level rod is a scientific instrument and must be treated as such; not only during use but also during storage and transporting.
  - When not in use they must be stored in their shipping containers to avoid damage.
  - The footpiece should be inspected frequently to make sure it has not been bent or otherwise damaged.

- d. Leveling monumentation.
  - All benchmarks used to monument Second-Order level lines should conform to criteria published in EM 1110-1-1002 (Survey Markers and Monumentation).
  - Benchmarks used to monument Second-Order level lines should be standard USACE brass caps set in concrete.
  - The concrete should be placed in holes deep enough to avoid local disturbance.
  - If the brass cap is not attached to an iron pipe, use some type of metal to reinforce the concrete prior to embedding the brass cap.
  - Concrete should be placed in a protected position.
  - If possible, benchmarks should be set close to a fence line, yet far enough away to permit plumbing of level rod.
  - Do not set monuments closer than four feet to a fence post, as the benchmark likely will be disturbed if the post is replaced.
  - Each brass cap must be stamped to identify it by the methods detailed in EM 1110-1-1002.
  - In addition to stamping a local number or name on the cap, it is optional to stamp the elevation on the brass cap after final elevation adjustment has been made.
  - The benchmarks must be set no less than 24 hours in advance of the level crew if the survey is to be made immediately after monument construction.

- e. Leveling notes.
  - Notes for Second-Order levels should be kept in a format approved by the District, or should follow recognized industry practice.
  - A set style cannot be developed due to different types of equipment that may be employed.
  - Elevations generally should not be carried in the field as they will be adjusted by the field office and closures approved prior to assigning a final adjusted elevation.
  - See the following section on Third-Order leveling for sample recording formats and sketches.

- f. Three-wire leveling.
  - This method can be used for most types of leveling work and will achieve any practical level of accuracy, including Second-Order.
  - However, most applications do not require the accuracies possible with three-wire leveling; plus, it is somewhat labor intensive.
  - Three-wire leveling can be applied if the reticule of the level has stadia lines and substadia that are spaced so that the stadia intercept is 0.3 ft at 100 feet, rather than the more typical 1.0 ft at 100 feet.
  - The substadia lines in instruments meant for three-wire leveling are short cross lines that cannot be mistaken for the long central line used for ordinary leveling.
  - Although there are many different observing techniques for three-wire leveling, in the following example, the rod is read at each of the three lines and the average is used for the final result.
  - Before each reading, the level bubble is centered.
  - The half-stadia intervals are compared to check for blunders.

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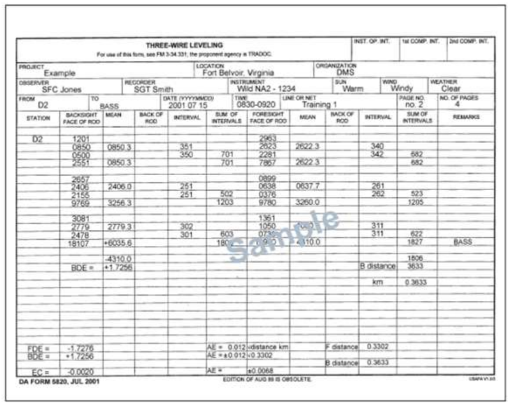
- The following values were taken and recorded and calculations made:

Upper Wire:	8.698	2.155 :Upper Interval
Middle Wire:	6.543	
Lower Wire:	4.392	2.151 :Lower Interval
Sum	19.633	
Average	6.544	

- The final rod reading would be 6.544 feet.
- The upper and lower intercepts differ by only 0.004 ft--an acceptable error for this sort of leveling and evidence that no blunder has been made.
- It is recommended that "Yard Rods" specifically designed for three-wire leveling operations be used instead of Philadelphia rods that are designed for ordinary leveling.

- A sample recording form is shown at the figure below.



THREE-WIRE LEVELING

For use of this form, see FM 3-34.331; the proponent agency is TRADOC.

INST. OF. INT. | 1st COMP. INT. | 2nd COMP. INT.

PROJECT: Example | LOCATION: Fort Belvoir, Virginia | ORGANIZATION: DMS

OBSERVER: SFC Jones | RECORDER: SGT Smith | INSTRUMENT: Wild NA2 - 1234 | SUN: Warm | WIND: Windy | WEATHER: Clear

FROM: D2 | TO: BASS | DATE (YYYYMMDD): 2001 07 15 | TIME: 0830-0920 | LINE OR NET: Training 1 | PAGE NO.: no. 2 | NO. OF PAGES: 4

STATION	BACKSIGHT FACE OF ROD	MEAN	BACK OF ROD	INTERVAL	SUM OF INTERVALS	FORESIGHT FACE OF ROD	MEAN	BACK OF ROD	INTERVAL	SUM OF INTERVALS	REMARKS
D2	1201					2963					
	0850	0850.3		351		2623	2622.3		340		
	0500			350	701	2281			342	682	
	2551	0850.3			701	7867	2622.3			682	
	2657					0899					
	2406	2406.0		251		0638	0637.7		261		
	2155			251	502	0376			262	523	
	9769	3256.3			1203	9780	3260.0			1205	
	3081					1361					
	2779	2779.3		302		1050	[illegible]		311		
	2478			301	603	[illegible]			311	622	
	18107	+6035.6			[illegible]	[illegible]	-4310.0			1827	BASS
		-4310.0								1806	
	BDE =	+1.7256							B distance	3633	
									km	0.3633	
FDE =	-1.7276				AE = 0.012	√distance km		F distance	0.3302		
BDE =	+1.7256				AE = ±0.012	√0.3302					
								B distance	0.3633		
EC =	-0.0020				AE =	±0.0068					

DA FORM 5820, JUL 2001 — EDITION OF AUG 89 IS OBSOLETE.

Figure 7: Three-wire leveling recording form--DA Form 5820.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

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- g. High accuracy differential leveling.
  - Methods and procedures for conducting highly precise (i.e. First-Order) differential leveling used for monitoring structural settlements are covered in EM 1110-2- 1009 (Structural Deformation Surveying).

Third-Order and Lower-Order Leveling

- a. General.
  - Leveling run for traverse profiles, temporary benchmarks, control of cross-sections, slope stakes, soundings, topographic mapping, structure layout, miscellaneous construction layout, and construction staking should be Third- or Fourth-Order leveling, unless otherwise directed.
  - All levels should originate from and tie into existing control; preferably from two or more benchmarks.
  - No level line should be left unconnected to control unless by specific instructions of the survey supervisor or written directive.

- b. Leveling accuracy.
  - The required accuracy (in feet) for Third-Order levels is 0.050 times the square root of M feet, and where "M" is the length of the level line in miles.
  - Construction Layout level work will conform to 0.100 times the square root of M feet.
  - The length of the line may be determined from quad sheets or larger scale map if a direct measure between points is not available.

- c. Leveling equipment.
  - The type of equipment needed is dependent upon the accuracy requirements.
  - (1) Third-Order level.
    - A semi-precise level can be used for Third-Order leveling, such as the tilting Dumpy type, three-wire reticule, or equivalent.
  - (2) Leveling rods.
    - The rods should be graduated in feet, tenths and hundreds of feet.
    - The Philadelphia rod or its equivalent is acceptable.
    - However, the project specifications will sometimes require that semi-precise rods be used that are graduated on the front in centimeters and on the back in half-foot intervals.
    - The Zeiss stadia rod, fold type, or its equivalent should be used when the specifications require semi-precise rods.

  - (3) Lower-order instruments.
    - The type of spirit level instrument used should ensure accuracy in keeping with required control point accuracy.
    - Precision levels are not required on lower-order leveling work.
    - The Fennel tilting level, dumpy level, Wye level, or their equivalent, are examples of levels that can be used.
    - A stadia rod with least readings of 0.05 ft or 1 cm will be satisfactory.
    - The use of turning pins and/or plates will depend upon the type of terrain or if rods may be placed on firm stones or roadways.

- d. Leveling monumentation.
  - The level line should be tied to all existing benchmarks along or adjacent to the line section being run.
  - In the event there are no existing benchmarks near the survey, new ones should be set, not more than 0.5 mile apart.
  - Steep landscape in the area of survey may require monuments to be set at a closer spacing.
  - Benchmarks should be set on permanent structures, such as, head walls, bridge abutments, pipes, etc.
  - Large spikes driven into the base of trees, telephone poles, and fence posts are acceptable for this level of work.
  - All temporary benchmarks must have a full description including location.
  - Unless they are on a turn, they are not considered temporary benchmarks.
  - No closures shown by an intermediate shot will be accepted.
  - All temporary benchmarks must have a name or number for future identification.

- e. Leveling equipment.
  - Turning pins should be driven into in the ground until rigid with no possibility of movement.
  - Turning points or temporary benchmarks will have a definite high point so that any person not familiar with the point will automatically hold the rod on the highest point, and so that it can spin free.
  - If solid rocks are being used for turns they must be marked with crayon or paint prior to taking readings.
  - It is not mandatory to use targets on the rod when the reading is clearly visible.
  - However, they are required in dense brush, when using grade rods, or when unusually long shots are necessary.
  - Rod bubbles should always be used to ensure the rod is held plumb.

- f. Leveling notes and sketches.
  - Complete notations or sketches should be made to identify level lines and side shots.
  - All Second- and Third-Order or lower level notes should be completely reduced in the field as the levels are run; with the error of closure noted at all tie in points.
  - In practice, the circuit will be corrected to true at each tie in point unless instructed to do otherwise by the survey supervisor or written directive.
  - Any change in rod reading should be initialed and dated so there is no doubt as to when a correction was made. Cross out erroneous readings--never erase them.
  - The instrument man should take care to keep peg notes on all turns in the standard field book.
  - The notes should be dated and noted as to what line is being run, station occupied, identification of turns, etc.
  - A complete description of each point on which an elevation is established should be recorded in the field book adjacent to the station designation.
  - Entries should be made in the book that give the references to the notes and other existing data used for elevations (e.g., TRAVERSE BOOK XXXX PAGE XX, USGS Quad XXXXXX, NOS Chart XXXX, etc.).
  - Level notes should conform to a standard industry format, e.g., Point--(+)BS-- HI--(-)FS--Elev.

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- In general, level notes should follow the formats shown in the following figures.

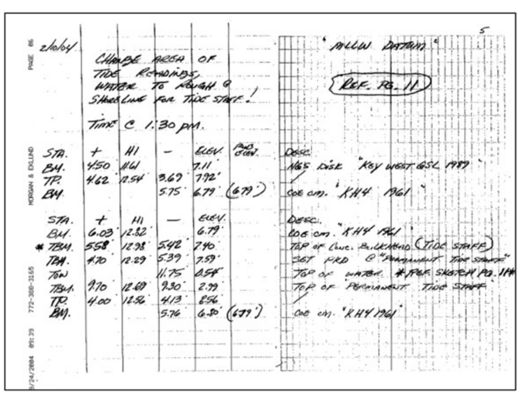


Figure 8: Sample single-wire level notes to set tide staff.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

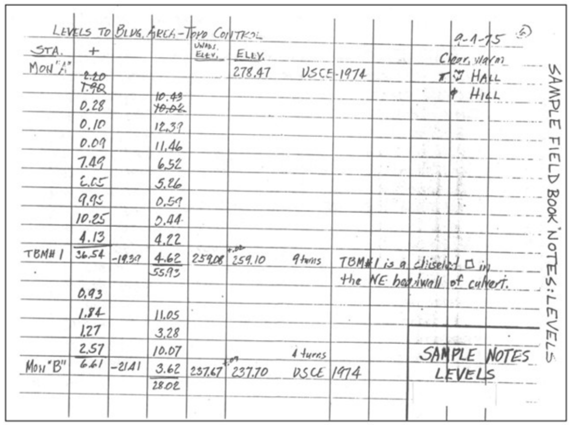


Figure 9: Sample level notes for bringing in vertical control to a project.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

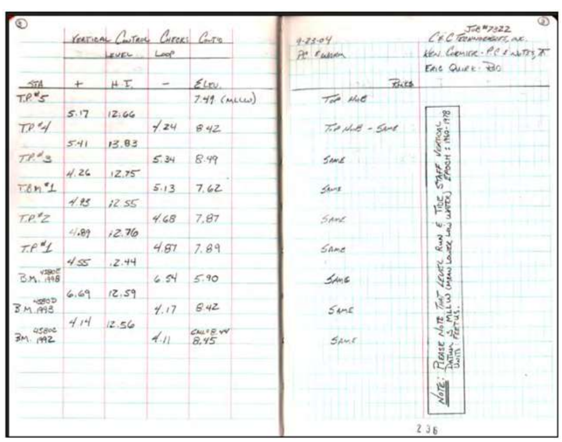
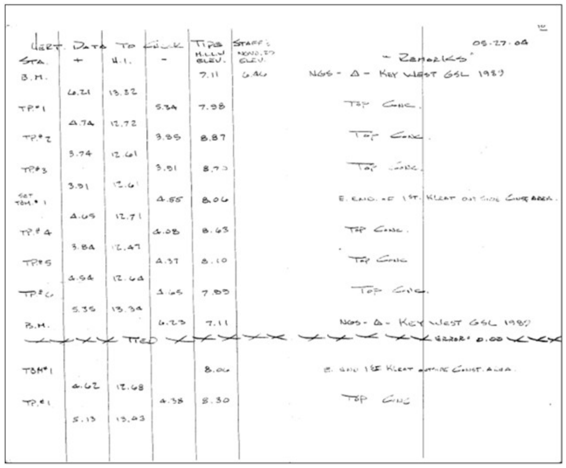


Figure 10: Sample single-wire level notes tying in tidal benchmarks on a dredging project.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)



Vert. Data To Check Tides STA.	+	H.I.	−	H.L.L.W. Elev.	Staff NGVD 29 Elev.	Remarks 05.27.04
B.M.				7.11	6.46	NGS - Δ - Key West GSL 1989
	6.21	13.32				
TP#1			5.34	7.98		Top Conc.
	4.74	12.72				
TP#2			3.85	8.87		Top Conc.
	3.74	12.61				
TP#3			3.91	8.70		Top Conc.
	3.91	12.61				
Set TBM#1			4.55	8.06		E. End of 1st Kleat outside Const. area.
	4.65	12.71				
TP#4			4.08	8.63		Top Conc.
	3.84	12.47				
TP#5			4.37	8.10		Top Conc.
	4.54	12.64				
TP#6			4.65	7.99		Top Conc.
	5.35	13.34				
B.M.			6.23	7.11		NGS - Δ - Key West GSL 1989
TIED						Error: 0.00
TBM#1				8.06		E. End 1st Kleat outside Const. area.
	4.62	12.68				
TP#1			4.38	8.30		Top Conc.
	5.13	13.43				

Figure 11: Standard single-wire level notes.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- However, local variations are acceptable--see Kavanagh 1997 for examples of different types of level notes.
- Level line sketches should be drawn in the field, as shown in the example at the figure below.

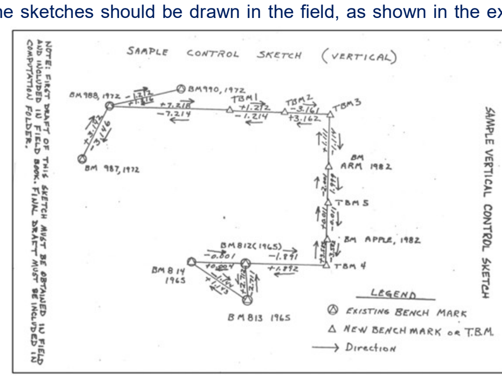


Figure 12: Sample vertical control project sketch showing forward and backward level runs for each leg.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- These sketches are particularly helpful in resolving complicated loop closures or where redundant lines have been run.
- Sketches may be drawn in a field book or on graph paper.
- They need not be at any particular orientation or scale.
- Original field notes or copies of notes may be submitted, as directed.

Calibrations and Adjustments

- To maintain the required accuracy, certain tests and adjustments must be made at prescribed intervals to both the levels and rods being used.
- a. Determination of stadia constant.
  - The stadia constant factor of the leveling instrument should be determined by calibration.
  - The stadia factor is required for measurement and computation of distances from the instrument to the leveling rod.
  - This determination is made independently for each level used in the field and is permanently recorded and kept with project files.
  - The determination is made by comparing the measured stadia distance to known distances on a test course.

Figure 13: C-factor calibration procedure.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- b. Determination of "C" Factor.
  - Each day, just before the leveling is begun, or just after the beginning of the day's observations, and immediately following any instance in which the level is subjected to unusual shock, the error of the level, or "C" factor, must be determined.
  - This determination can be made during the regular course of leveling or over a special test course; in either case the recording of the observations must be done on a separate page of the recording notes with all computations shown.
  - If the determination is made during the first setup of the regular course of levels, the following procedure is used.
  - After the regular observations at the instrument station "A" are completed, transcribe the last FS reading "a" as part of the error determination; call up the backsight rodman and have the rod placed about 10 meters from the instrument; read the rod "b" over the instrument to a position "B" about 10 meters behind the front rod; read the front rod "c" and then the back rod "d".
  - The two instrument stations must be between the rod points.
  - The readings must be made with the level bubble carefully centered and then all three wires are read for each rod reading.

- The required "C" factor determined is the ratio of the required rod reading correction to the corresponding subtended interval, or:

$$C = (R1 - R2) / (R3 - R4) \qquad (2)$$

- where
  - R1 = Sum near rod readings
  - R2 = Sum distant rod readings
  - R3 = Sum distant rod readings
  - R4 = Sum near rod readings

- The total correction for curvature and refraction must be applied to each distant rod reading before using them in the above formula.
- It must be remembered that the sum of the rod intervals must be multiplied by the stadia constant in order to obtain the actual distance before correction.
- The maximum permissible "C" factor varies with the stadia constant of the instrument.
- The instruments must be adjusted if the "C" factor is:

$$C > 0.004 \quad \text{for a stadia constant of } 1/100$$
$$C > 0.007 \quad \text{for a stadia constant of } 1/200$$
$$C > 0.010 \quad \text{for a stadia constant of } 1/333.$$

- The determination of the "C" factor should be made under the expected conditions of the survey as to length of sight, character of ground, and elevation of line of sight above the ground.
- The date and time must be recorded for each "C" factor determination, since this information is needed to compute leveling corrections.

Collimation Check

Project: Example	Location: Fort Belvoir, Virginia	Organization: DMS			
Observer: SFC Jones	Recorder: SGT Smith	Instrument: WILD NA2-1234	Sun: Clear	Wind: Calm	Weather: Warm
From / To	Date: 2000 09 30	Time: 0813-0835	Line or Net: Belvoir Net 1	Page No.	No. of Pgs.

Station	Backsight Face of rod	Mean	Back of rod	Interval	Sum of Intervals	Foresight Face of rod	Mean	Back of rod	Interval	Sum of Intervals	Remarks
	1821					1055					STADIA
	1771	1771.0		050		0680	0679.7		375		CONSTANT
	1721			050	100	0304			376	751	0.100
	5313	1771.0			100	2039	0679.7			751	
	1476					2908					
	1432	1432.0		044		2530	2530.7		378		
	1388			044	088	2154			376	754	
	9609	3203.0			188	9631	3210.4			1505	
						C & R #1	-0.4		-	188	
MEAN MIDDLE WIRE						C & R #2	-0.4		SUM (1)	1317	
SUM			SUM MEAN MIDDLE WIRE (FORESIGHT)				-3209.6				
ROD CORRECTIONS			SUM MEAN MIDDLE WIRE (BACKSIGHT)				3203.0			±	
CURVATURE & REFRACTION					SUM (1)	1317	-6.6		C =	-.0050	
NOTE:										+754	
CORRECTION IS IN ROD										-3.8	[illegible]
UNITS										+2530.0	
										2526.2	[illegible]
SIGHT DISTANCE										2.526	

METERS	YARDS	C & R
00	00	
27.0	28.2	00
46.8	48.9	0.1
60.4	63.1	0.2
71.4	74.7	0.3
81.0	84.7	0.4
89.5	93.6	0.5
97.3	101.8	0.6
104.5	109.3	0.7
111.3	116.4	0.8

Sample

FOR CLASSROOM PURPOSES ONLY

INST OP INT

1st COMP INT

2nd COMP INT

DMS Form 5820-R, JAN 97

Figure 14: Example of C-factor computation.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- c. Adjustment of level.
  - The type of instrument being used will dictate the method and procedure used to adjust the instrument if the "C" factor exceeds the allowable limits.
  - The manufacturer's procedures should be followed when adjusting a level.
- d. Test of rod levels.
  - Precise rod levels must be tested once each week during regular use--or whenever they receive a severe shock.
  - This test is made with the level rod bubble held at its center, and the deviation of the face and edge of the rod from the vertical are determined.
  - If the deviation from the vertical exceeds 0.01 meter on a 3-meter length of rod, the rod level must be adjusted.
  - The rod level is adjusted in the same manner as any other circular bubble.
  - A statement must be inserted in the records showing the manner in which the test was made, the error that was found, if any, and whether an adjustment was made.
  - When using other than precise leveling rods, this test is not required.

Care of Level Instruments

- The following sections on care and maintenance of leveling equipment are taken from the CALTRANS Surveys Manual.
- This guidance is applicable to Corps field operations.
- a. Optical Pendulum Level.
  - Pendulum levels are fast, accurate, and easy to maintain.
  - Proper care and service is required to provide continuous service and to maintain precision in measurement.
  - Never disassemble an instrument in the field.
  - Only make those adjustments outlined in the operator's manual.

- Care of a Pendulum Level:
  - To prevent compensator damage, do not spin, bounce, or hit pendulum levels.
  - Protect the level from dust.
    - Dust or foreign matter inside the scope can cause the compensator damping device to hang up.

- b. Circular bubble test and adjustment.
  - Frequently check adjustment of the bullseye bubble.
  - Adjust the bubble to the center of the bullseye.
  - Make certain the bubble is adjusted along the line of sight and 90° to the line of sight as well.
  - Proper adjustment reduces the possibility of compensator hang-up.
  - Adjustment will be easier if done in the shade, where temperature is constant.

- c. Horizontal cross-hair test and adjustment (Two-Peg Test).
  - At least once every 90 days or when discrepancies show up in the leveling work and before every three-wire level survey, the "Two-Peg Test" should be performed as follows:
    - Select two benchmarks "A" and "B" approximately 60 m apart.
    - Set up the level midway between the two points "A" and "B" and record the rod readings of each point determining their difference in elevation.
    - Move the level 6 m beyond either benchmark and record the rod reading of both points again, once again determining their difference in elevation.
    - If the difference in elevation measured at each setup is the same, the level is in adjustment.
      - If not, the horizontal cross-hair should be adjusted as detailed in the operator's manual.
    - After the adjustment repeat the "peg test" again to check the adjustment.

- d. Mechanical functions.
  - To check for compensator hang-up, lightly tap the telescope with a pencil or operate the fine movement screw jerkily to and fro.
  - If the compensator is slow to respond or malfunctioning, send the instrument to an approved repair service.
  - There are no mechanical field adjustments that can be made on the compensator.

- e. Electronic Digital Bar-Code System.
  - Digital bar-code levels operate by comparing the observed digital image of a bar-code leveling rod with a map of the bar code stored in the level's memory.
  - These instruments are also equipped with a conventional pendulum-type compensator and may be used as an optical level.
  - An on-board computer processes all leveling operations including determination of sight lengths.
  - A bar-code system should include:
    - Digital level with data recorder module or cable connected data collector
    - Data reader and/or appropriate computer interface
    - Bar-code leveling rods

Figure 15: Digital Bar Code Level--Wild NA2.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- (1) Field operations.
  - At the beginning and end of each day's operation, check the instrument for collimation error, recording the tests into the survey notes.
  - When using electronic digital leveling instruments, the absolute collimation error will be recorded along with the leveling data.
  - If an error in excess of 2 mm within a 60 m sight distance is detected, the level should be readjusted.
  - If the instrument is severely jolted or bumped, or suspected as such, it should be immediately checked.
  - Manufacturers' specifications state that the electronic digital leveling instrument should not be exposed to direct sunlight and recommend use of an umbrella in bright sunlight.

- (2) Horizontal cross-hair test and adjustment (Two-Peg Test).
  - The test and adjustment procedure for an electronic digital level is geometrically similar to the two peg procedure for a conventional optical level.
  - However, all horizontal and vertical measurements and differences are measured and recorded electronically.
  - The collimation error is recorded by the on-board computer.

- (3) Data collection, storage, and transfer.
  - Raw data generated by an electronic digital level is stored in a data collector and processed into field book format.
  - Software will perform simple or least-squares adjustment of the data.
  - An ASCII file may be created that can be imported into road design software.

- (4) Leveling rods.
  - Leveling rods should be maintained and checked as any other precision equipment.
  - Accurate leveling depends as much on the condition of the rods as on the condition of the levels.
  - Reserve an older rod for rough work, such as measuring inverts, mud levels, water depths, etc.

Routine Maintenance and Care of Level Rods

- a. Maintenance procedures common to all types of rods are.
  - Periodically lubricate hardware and slip joints with an oil-free silicon spray.
  - Clean sand and grit from slip joints.
  - Clean graduated faces with a damp cloth and wipe dry.
  - Keep the base plate clean.
  - Periodically check all screws and hardware for snugness and operation.
  - Periodically check accuracy by extending rod and measuring between graduations across rod section divisions with an accurate tape.

- b. Transport and storage.
  - If possible, leave a wet rod uncovered and extended until it is thoroughly dry.
  - Store rods in protective sleeves or cases, in a dry location, either vertically (not leaning), or horizontally.
    - When stored horizontally, either fully support the rod or provide at least three-point support.

- c. Field operations.
  - Touch graduated faces only when necessary and avoid laying the rod where the graduated face will come into contact with other tools, objects, or materials that could mar the face.
  - Do not abuse a rod by throwing, dropping, dragging, or placing it where it might fall.
  - Do not lay a rod in sand, dust, or loose granular material.
  - Lower rod sections as the rod is being collapsed.
    - Do not let them fall or drop.

- d. Direct reading rod.
  - At frequent intervals, check all components for wear.
  - Periodically lubricate all hardware, racks, and rollers with an oil-free silicone aerosol spray.
  - If the tape guides begin to snag or bind the tape, have the rod repaired.

- e. Fiberglass leveling rod.
  - Dowels through the bottom of each section keep the section above from falling inside the lower section.
  - Dropping the sections when collapsing the rod will loosen the dowels causing the sections to jam and may also shatter the fiberglass around the dowel holes.
  - Observe the following precautions:
    - When the slip joint goes bad, remove the rod from service.
    - Lubricate fiberglass rods with an oil-less silicone spray or with talcum.

- f. Invar leveling rods.
  - Invar rods are precisely made and standardized; extra care is required to maintain this precision.
  - Observe the following precautions:
    - Store, fully supported and stopped, in a water-proof case.
    - Do not use invar rods in rain or dust.
    - Carry parallel with the ground, in alternate "face-up" and "face-down" positions to equalize weight stresses.
    - Avoid laying an invar rod on the ground.
    - If foreign matter has "fouled" a rod, carefully disassemble and clean.
    - The rod tape must slide freely in the recessed guides as the wooden staff swells or shrinks.

- g. Bar-code leveling staffs (Rods).
  - A typical bar-code leveling staff is of a different design and construction than a conventional level rod.
  - Several types of bar-code rods are available, depending on the type of work performed.
  - Designs range from an aluminum/invar-tape, precise staff to various sectional staffs constructed of wood, aluminum, or fiberglass.
  - Care and maintenance of these staffs is minimal due to their simplistic construction.
  - Store in clean, dry condition and always transport in carrying cases.

Maintenance of Survey Instrument Accessories

- a. Tripods.
  - Tripods support and provide a fixed base for all types of surveying instruments.
  - The typical tripod has a 5/8-in. x 11 thread fastener to secure an instrument or accessory to the tripod head.
  - The head provides a lateral adjustment range for the instrument of approximately 25 mm.
  - The tripods are of a wide-frame design and have extendible legs.
  - A secure and stable tripod is required for the support of precision instruments.
  - There should be no slack between the various components of a tripod.
  - Loose joints or fittings will cause instability.

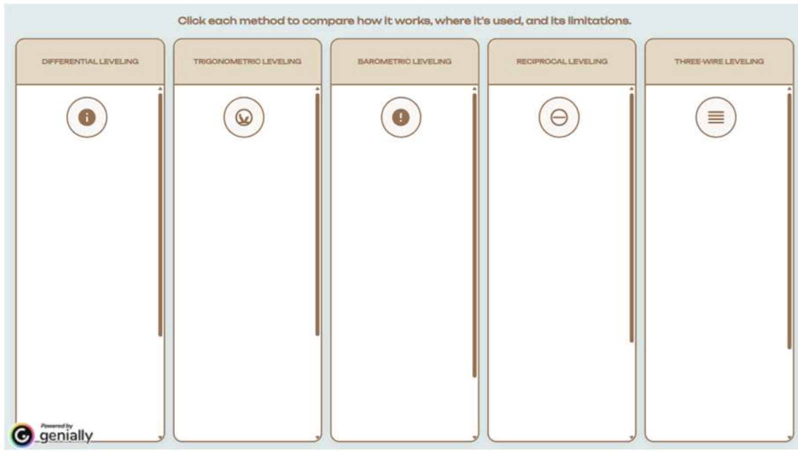
- Some guidelines to properly maintain tripods are:
  - Maintain a firm snugness in all metal fittings.
    - Over-tightening is the cause of crushed wooden components and stripped threads.
  - Tighten leg hinges just enough to support the fully extended legs when a tripod is lifted clear of the ground.
  - Keep the metal tripod shoes tight and free of dirt and debris.
  - Keep wooden parts of tripods well painted or varnished to reduce swelling and shrinking due to moisture content of the wood.
  - Always replace top caps when tripod is not in use to protect the mounting surface and head from damage.
  - Use care when placing or removing tripods from the survey vehicle, as significant damage can occur.
    - Ensure that carry compartments are designed and constructed to isolate tripods from each other and from other equipment.

- b. Tribrachs.
  - Tribrachs are the detachable base for most survey instruments and many accessories.
  - They are equipped with an optical plummet and spherical "bullseye" level.
  - The ability to "leapfrog" instrument setups by interchanging instruments, prisms, targets, or antennas without disturbing the setup of a tribrach greatly enhances the speed, efficiency, and accuracy of a survey.
  - Some guidelines to properly maintain tribrachs are:
    - Transport tribrachs in separate compartments or containers to prevent damage to the base surfaces, spherical level, and optical plummet.
    - Do not over-tighten the tripod fastener screw.
    - Clean leveling screws regularly.
    - When tribrachs are not in use, set leveling screws at mid-range, usually marked by a horizontal line.
    - Use care whenever using range poles mounted on a tribrach to vertically extend a sight, antenna, or prism.
      - Extensions place considerable stress on the leveling plate.
    - Adjust spherical level and vertical collimation of optical plummet routinely.

- c. Prism Poles/Antenna Poles.
  - An attached adjustable spherical level bubble (bullseye) is used to maintain a prism/antenna pole in a vertical position.
  - A maladjusted level bubble may cause systematic error when using the pole.
  - A simple method for checking the accuracy of the bullseye bubble is to check the rod by placing it against a door jamb or other permanent vertical part of a building that has been previously verified as being vertical.

Check next slide to explore an interactive GIF about The Leveling Method Selector

PRIMARY CONTROL SURVEYS FOR PROJECT MAPPING (CONT'D)



ACCURACY STANDARDS FOR ENGINEERING, CONSTRUCTION, AND FACILITY MANAGEMENT CONTROL AND TOPOGRAPHIC SURVEYS

Purpose

- This chapter sets forth accuracy standards and other related criteria that are recommended for use in largescale site plan topographic surveys for engineering and construction purposes.
- These standards relate to surveys performed to locate, align, and stake out construction for civil and military projects, e.g., buildings, utilities, roadways, runways, flood control and navigation projects, training ranges, etc.
- In many cases, these engineering surveys are performed to provide the base horizontal and vertical control used for area mapping, GIS development, preliminary planning studies, detailed site plan drawings for construction plans, construction measurement and payment, preparing as-built drawings, installation master planning mapping, future maintenance and repair activities, and other AM/FM products.
- Most engineering surveying standards currently used are based on local practice, or may be contained in State minimum technical standards.
- The standards given in this chapter conform to the criteria prescribed in EM 1110-1-2909 (Geospatial Data and Systems) and the FGDC Geospatial Positioning Accuracy Standard.

General Surveying and Mapping Specifications

- Construction plans, maps, facility plans, and CADD/GIS databases are created by a variety of terrestrial, satellite, acoustic, or aerial mapping techniques that acquire planimetric, topographic, hydrographic, or feature attribute data.
- Specifications for obtaining these data should be "performance-based" and not overly prescriptive or process oriented.
- They should be derived from the functional project requirements and use recognized industry accuracy standards where available.

- a. Industry standards.
  - Maximum use should be made of industry standards and consensus standards established by private voluntary standards bodies--in lieu of Government-developed standards.
  - Therefore, industry-developed accuracy standards should be given preference over Government standards.
  - A number of professional associations have published surveying and mapping accuracy standards, such as the American Society for Photogrammetry and Remote Sensing (ASPRS), the American Society of Civil Engineers (ASCE), the American Congress on Surveying and Mapping (ACSM), and the American Land Title Association (ALTA).
  - When industry standards are non-existent, inappropriate, or do not meet a project's functional requirement, FGDC, DOD, DA, or USACE standards may be specified as criteria sources.
  - Minimum technical standards established by state boards of registration, especially on projects requiring licensed surveyors, should be followed when legally applicable.
  - Local surveying and mapping standards should not be developed where consensus industry standards or DOD/DA standards exist.

- b. Performance specifications.
  - Performance-oriented (i.e. outcome based) specifications are recommended in procuring surveying and mapping services.
  - Performance specifications set forth the end results to be achieved (final map format, data content, and/or accuracy standard) and not the means, or technical procedures, used to achieve those results.
  - Performance-oriented specifications typically provide the most flexibility and use of state-of-the-art instrumentation and techniques.
  - Performance specifications should succinctly define only the basic mapping requirements that will be used to verify conformance with the specified criteria, e.g., mapping limits, feature location and attribute requirements, scale, contour interval, map format, sheet layout, and final data transmittal, archiving or storage requirements, required accuracy criteria standards for topographic and planimetric features that are to be depicted, and quality assurance procedures.
  - Performance-oriented specifications should be free from unnecessary equipment, personnel, instrumentation, procedural, or material limitations; except as needed to establish comparative cost estimates for negotiated services.

- c. Prescriptive (procedural) specifications.
  - Use of prescriptive specifications should be kept to a minimum, and called for only on highly specialized or critical projects where only one prescribed technical method is appropriate or practical to perform the work.
  - Prescriptive specifications typically require specific field instrumentation, equipment, personnel, office technical production procedures, or rigid project phasing with on-going design or construction.
  - Prescriptive specifications may, depending on the expertise of the writer, reduce flexibility, efficiency, and risk, and can adversely impact project costs if antiquated methods or instrumentation are required.
  - Prescriptive specifications also tend to shift most liability to the Government.
  - Occasionally, prescriptive specifications may be applicable to Corps projects involving specialized work not routinely performed by private surveying and mapping firms, e.g., mapping tactical operation sites, mapping hazardous, toxic, and radioactive waste (HTRW) clean-up sites, military/tactical surveying, or structural deformation monitoring of locks, dams, and other flood control structures.

- d. Quality control and quality assurance.
  - Quality control (QC) of contracted surveying and mapping work should generally be performed by the contractor.
  - Therefore, USACE quality assurance (QA) and testing functions should be focused on whether the contractor meets the required performance specification (e.g., accuracy standard), and not the intermediate surveying, mapping, and compilation steps performed by the contractor.
  - The contractor's internal QC will normally include independent tests that may be periodically reviewed by the Government.
  - Government-performed (or monitored) field testing of map accuracies is an optional QA requirement, and should be performed when technically and economically justified, as determined by the ultimate project function.

- e. Metrication.
  - Surveying and mapping performed for design and construction should be recorded and plotted in the units prescribed for the project by the requesting Command or project sponsor.
  - During transition to the metric system, inch-pound (IP) units or soft conversions may be required for some geospatial data.

- f. Spatial coordinate reference systems.
  - Where practical, feasible, or applicable, civil and military projects should be adequately referenced to nationwide or worldwide coordinate systems directly derived from, or indirectly connected to, GPS satellite observations.
  - In addition, navigation and flood control projects in tidal areas should be vertically referenced to the latest datum epoch established by the Department of Commerce.

Accuracy Standards for Engineering and Construction Surveying

- a. Accuracy standards.
  - Engineering and construction surveys are normally specified and classified based on the horizontal (linear) point closure ratio or a vertical elevation difference closure standard.
  - This type of performance criteria is most commonly specified in Federal agency, state, and local surveying standards, and should be followed and specified by USACE commands.
  - These standards are applicable to most types of engineering and construction survey equipment and practices (e.g., total station traverses, differential GPS, differential spirit leveling).
  - These accuracy standards are summarized in the following tables.

USACE Classification	Closure Standard	
Engr & Const Control	Distance (Ratio)	Angle (Secs)
First-Order	1:100,000	$2\cdot\sqrt{N}$ [1]
Second Order, Class I	1:50,000	$3\cdot\sqrt{N}$
Second Order, Class II	1:20,000	$5\cdot\sqrt{N}$
Third Order, Class I	1:10,000	$10\cdot\sqrt{N}$
Third Order, Class II	1: 5,000	$20\cdot\sqrt{N}$
Engineering Construction (Fourth-Order)	1: 2,500	$60\cdot\sqrt{N}$

[1] N = Number of angle stations

Table 1: Minimum Closure Accuracy Standards for Engineering and Construction Surveys.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

USACE Classification	Elevation Closure Standard (ft)[1]	(mm)
First-Order, Class I	$0.013\cdot\sqrt{M}$	$3\cdot\sqrt{K}$
First-Order, Class II	$0.017\cdot\sqrt{M}$	$4\cdot\sqrt{K}$
Second Order, Class I	$0.025\cdot\sqrt{M}$	$6\cdot\sqrt{K}$
Second Order, Class II	$0.035\cdot\sqrt{M}$	$8\cdot\sqrt{K}$
Third Order	$0.050\cdot\sqrt{M}$	$12\cdot\sqrt{K}$
Construction Layout	$0.100\cdot\sqrt{M}$	$24\cdot\sqrt{K}$

[1] $\sqrt{M}$ or $\sqrt{K}$ = square root of distance in Miles or Kilometers

Table 2: Minimum Elevation Closure Accuracy Standards for Engineering and Construction Surveys.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- b. Survey closure standards.
  - Survey closure standards listed in the previous tables should be used as a basis for classifying, standardizing, and evaluating survey work.
  - The point and angular closures (i.e. traverse misclosures) relate to the relative accuracy derived from a particular survey.
  - This relative accuracy (or, more correctly, precision) is estimated based on internal closure checks of a traverse survey run through the local project, map, land tract, or construction site.
  - Relative survey accuracy estimates are always expressed as ratios of the traverse/loop closure to the total length of the survey (e.g., 1:10,000).

- (1) Horizontal closure standard.
  - The horizontal point closure ratio 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 traverse or closed loop survey, these angular misclosures should be distributed (balanced) before assessing positional misclosure.
  - In cases where differential GPS vectors are measured in three-dimensional geocentric coordinates, then the horizontal component of position misclosure is assessed relative to Table 1.

- (2) 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 millimeters or feet) should not exceed the limits shown in Table 2, where the line or circuit length is measured in the applicable units.
  - Fourth-Order accuracies are intended for construction layout grading work.

- c. Construction survey accuracy standards.
  - Construction survey procedural and accuracy specifications should follow recognized industry and local practices.
  - General procedural guidance is contained in a number of standard commercial texts--e.g., Kavanagh 1997.
  - Accuracy standards for construction surveys will vary with the type of construction, and may range from a minimum of 1:2,500 up to 1:20,000.
  - A 1:2,500 "4th-Order Construction" classification is intended to cover temporary control used for alignment, grading, and measurement of various types of construction, and some local site plan topographic mapping or photo mapping control work.
  - Lower accuracies (1:2,500-1:5,000) are acceptable for earthwork, dredging, embankment, beach fill, and levee alignment stakeout and grading, and some site plan, curb and gutter, utility building foundation, sidewalk, and small roadway stakeout.
  - Moderate accuracies (1:5,000) are used in most pipeline, sewer, culvert, catch basin, and manhole stakeouts, and for general residential building foundation and footing construction, and highway pavement.
  - Somewhat higher accuracies (1:10,000-1:20,000) are used for aligning longer bridge spans, tunnels, and large commercial structures.
  - For extensive bridge or tunnel projects, 1:50,000 or even 1:100,000 relative accuracy alignment work may be required.
  - Grade elevations are usually observed to the nearest 0.01 ft for most construction work, although 0.1 ft accuracy is sufficient for riprap placement, earthwork grading, and small diameter pipe placement.
  - Construction control points are usually marked by semi permanent or temporary monuments (e.g., plastic hubs, P-K nails, iron pipes, wooden grade stakes).
  - Construction control is usually set from existing boundary, horizontal, and vertical control points.

- d. Geospatial positioning accuracy standards.
  - Many control surveys are now being efficiently and accurately performed using radial (spur) techniques--e.g., single line vectors from electronic total stations or kinematic differential GPS to monumented control points, topographic feature points, property corners, etc.
  - Since these surveys may not always result in loop closures (i.e. closed traverse) alternative specifications for these techniques must be allowed.
  - This is usually done by specifying a radial positional accuracy requirement.
  - The required positional accuracy may be estimated based on the accuracy of the fixed reference point, instrument, and techniques used.
  - Ratio closure standards in Tables 1 and 2 may slowly decline as more use is made of nation-wide augmented differential GPS positioning and electronic total station survey methods.

- (1) GPS satellite positioning technology allows development of map features to varying levels of accuracy, depending on the type of equipment and procedures employed.
  - Government and commercial augmented GPS systems allows direct, real-time positioning of static AM/FM type features and dynamic platforms (survey vessels, aircraft, etc.).
  - Site plan drawings, photogrammetric control, and related GIS features can be directly constructed from GPS or differential GPS observations, at accuracies ranging from 1 cm to 20 meters (95%).

- (2) Accuracy classifications of maps and related GIS data developed by GPS methods can be estimated based on the GPS positioning technique employed.
  - Permanent GPS reference stations (Continuously Operating Reference Stations or CORS) can provide centimeter-level point positioning accuracies over wide ranges; thus providing direct map/feature point positioning without need for preliminary control surveys.

- e. Higher-order surveys.
  - Requirements for relative line accuracies exceeding 1:50,000 are rare for most facility engineering, construction, or mapping applications.
  - Surveys requiring accuracies of FirstOrder (1:100,000) or better (e.g., A- or B-Order) should be performed using FGDC geodetic standards and specifications.
  - These surveys must be adjusted and/or evaluated by the National Geodetic Survey (NGS) if official certification relative to the national network is required.

- f. Instrumentation and field observing criteria.
  - In accordance with the policy to use performance-based standards, rigid prescriptive requirements for survey equipment, instruments, or operating procedures are discouraged.
  - Survey alignment, orientation, and observing criteria should rarely be rigidly specified; however, general guidance regarding limits on numbers of traverse stations, minimum traverse course lengths, auxiliary azimuth connections, etc., may be provided for information.
  - For some highly specialized work, such as dam monitoring surveys, technical specifications may prescribe that a general type of instrument system be employed, along with any unique operating, calibration, or recordation requirements.

- g. Connections to existing control.
  - Surveys should normally be connected to existing local control or project control monuments/benchmarks.
  - These existing points may be those of any Federal (including Corps project control), State, local, or private agency.
  - Ties to local Corps or installation project control and boundary monuments are absolutely essential and critical to design, construction, and real estate.
  - In order to minimize scale or orientation errors, at least two existing monuments should be connected.
  - It is recommended that Corps surveys be connected with one or more stations on the National Spatial Reference System (NSRS), when practicable and feasible.
  - Connections with local project control that have previously been connected to the NSRS are normally adequate in most cases.
  - Connections with the NSRS shall be subordinate to the requirements for connections with local/project control.

- h. Survey computations, adjustments, and quality control/assurance.
  - Survey computations, adjustments, and quality control should be performed by the organization responsible for the actual field survey.
  - Contract compliance assessment of a survey should be based on the prescribed point closure standards of internal loops, not on closures with external networks of unknown accuracy.
  - In cases where internal loops are not observed, then assessment must be based on external closures.
  - Specifications should not require closure accuracy standards in excess of those required for the project, regardless of the accuracy capabilities of the survey equipment.
  - Least-squares adjustment methods should be optional for Second-Order or lower-order survey work.
  - Details on network adjustments are covered in EM 1110-1-1003 (NAVSTAR GPS Surveying).
  - Professional contractors should not be restricted to rigid computational methods, software, or recording forms.
  - Use of commercial software adjustment packages is strongly recommended.

- i. Data recording and archiving.
  - Field survey data may be recorded and submitted either manually or electronically.
  - Manual recordation should follow standard industry practice, using field book formats outlined in various technical manuals.

Accuracy Standards for Maps and Related Geospatial Products

- Map accuracies are defined by the positional accuracy of a particular graphical or spatial feature depicted.
- A map accuracy standard classifies a map as statistically meeting a certain level of accuracy.
- For most engineering projects, the desired accuracy is stated in the specifications, usually based on the final development scale of the map--both the horizontal "target" scale and vertical relief (specified contour interval or digital elevation model).
- Often, however, in developing engineering plans, spatial databases may be developed from a variety of existing source data products, each with differing accuracies--e.g., mixing 1 inch = 60 ft topo plans with 1 inch = 400 ft reconnaissance topo mapping.
- Defining an "accuracy standard" for such a mixed database is difficult and requires retention (attribution) of the source of each data feature in the base.
- In such cases the developer must estimate the accuracy of the mapped features.

- a. ASPRS Standard.
  - For site mapping of new engineering or planning projects, there are a number of industry and Federal mapping standards that may be referenced in contract specifications.
  - The recommended standard for facility engineering is the ASPRS "Accuracy Standards for Large Scale Maps".
  - This standard, like most other mapping standards, defines map accuracy by comparing the mapped location of selected well-defined points to their "true" location, as determined by a more accurate, independent field survey.
  - Alternately, when no independent check is feasible or practicable, a map's accuracy may be estimated based on the accuracy of the technique used to locate mapped features-- e.g., photogrammetry, GPS, total station, plane table.

- The ASPRS standard has application to different types of mapping, ranging from wide-area, small-scale, GIS mapping to large-scale construction site plans.
- It is applicable to all types of horizontal and vertical geospatial mapping derived from conventional topographic surveying or photogrammetric surveys.
- This standard may be specified for detailed construction site plans that are developed using conventional ground topographic surveying techniques (electronic total stations, plane tables, kinematic GPS).
- The ASPRS standard is especially applicable to site plan development work involving mapping scales larger than 1:20,000 (1 inch = 1,667 ft); it therefore applies to the more typical engineering map scales in the 1:240 (1 inch = 20 ft) to 1:4,800 (1 inch = 400 ft) range.
- Its primary advantage over other standards is that it contains more definitive statistical map testing criteria, which, from a contract administration standpoint, is desirable.
- Using the guidance in Tables 3 and 4 below, specifications for site plans need only indicate the ASPRS map class, target scale, and contour interval.

Target Map Scale	ASPRS Limiting RMSE in X or Y (Meters)		
Ratio m/m	Class 1	Class 2	Class 3
1:50	0.0125	0.025	0.038
1:100	0.025	0.05	0.075
1:200	0.050	0.10	0.15
1:500	0.125	0.25	0.375
1:1,000	0.25	0.50	0.75
1:2,000	0.50	1.00	1.5
1:2,500	0.63	1.25	1.9
1:4,000	1.0	2.0	3.0
1:5,000	1.25	2.5	3.75
1:8,000	2.0	4.0	6.0
1:10,000	2.5	5.0	7.5
1:16,000	4.0	8.0	12.0
1:20,000	5.0	10.0	15.0
1:25,000	6.25	12.5	18.75
1:50,000	12.5	25.0	37.5
1:100,000	25.0	50.0	75.0
1:250,000	62.5	125.0	187.5

Table 3a: ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Meters) for Well-Defined Points.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

Target Map Scale		ASPRS Limiting RMSE in X or Y (Feet)		
1"= x ft	Ratio ft/ft	Class 1	Class 2	Class 3
5	1:60	0.05	0.10	0.15
10	1:120	0.10	0.20	0.30
20	1:240	0.2	0.4	0.6
30	1:360	0.3	0.6	0.9
40	1:480	0.4	0.8	1.2
50	1:600	0.5	1.0	1.5
60	1:720	0.6	1.2	1.8
100	1:1,200	1.0	2.0	3.0
200	1:2,400	2.0	4.0	6.0
400	1:4,800	4.0	8.0	12.0
500	1:6,000	5.0	10.0	15.0
800	1:9,600	8.0	16.0	24.0
1,000	1:12,000	10.0	20.0	30.0
1,667	1:20,000	16.7	33.3	50.0

Table 3b: ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Feet) for Well-Defined Points.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

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ASPRS Limiting RMSE in Meters

Target Contour Interval	Topographic Feature Points			Spot or Digital Terrain Model Elevation Points		
Meters	Class 1	Class 2	Class 3	Class 1	Class 2	Class 3
0.10	0.03	0.07	0.10	0.02	0.03	0.05
0.20	0.07	0.13	0.2	0.03	0.07	0.10
0.25	0.08	0.17	0.25	0.04	0.08	0.12
0.5	0.17	0.33	0.50	0.08	0.16	0.25
1	0.33	0.66	1.0	0.17	0.33	0.5
2	0.67	1.33	2.0	0.33	0.67	1.0
4	1.33	2.67	4.0	0.67	1.33	2.0
5	1.67	3.33	5.0	0.83	1.67	2.5
10	3.33	6.67	10.0	1.67	3.33	5.0

Table 4a: ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Meters).

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

ASPRS Limiting RMSE in Feet

Target Contour Interval	Topographic Feature Points			Spot or Digital Terrain Model Elevation Points		
ft	Class 1	Class 2	Class 3	Class 1	Class 2	Class 3
0.5	0.17	0.33	0.50	0.08	0.16	0.2
1	0.33	0.66	1.0	0.17	0.33	0.5
2	0.67	1.33	2.0	0.33	0.67	1.0
4	1.33	2.67	4.0	0.67	1.33	2.0
5	1.67	3.33	5.0	0.83	1.67	2.5

Table 4b: ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Feet).

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- b. Horizontal (planimetric) accuracy criteria.
  - The ASPRS planimetric standard compares the root mean square error (RMSE) of the average of the squared discrepancies, or differences in coordinate values between the map and an independent topographic ground survey of higher accuracy (i.e. a check survey).
  - The "limiting RMSE" is defined in terms of meters (feet) at the ground scale rather than in millimeters (inches) at the target map scale.
  - This results in a linear relationship between RMSE and target map scale--as map scale decreases, the RMSE increases linearly.
  - The RMSE is the cumulative result of all errors including those introduced by the processes of ground control surveys, map compilation, and final extraction of ground dimensions from the target map.
  - The limiting RMSE shown in Table 3 is the maximum permissible RMSE established by the ASPRS standard.
  - These ASPRS limits of accuracy apply to well-defined map test points only--and only at the specified map scale.

- c. Vertical (topographic) accuracy criteria.
  - Vertical accuracy has traditionally been, and currently still is, defined relative to the required contour interval for a map.
  - In cases where digital elevation models (DEM) or digital terrain models (DTM) are being generated, an equivalent contour interval can be specified, based on the required digital point/spot elevation accuracy.
  - The contours themselves may be later generated from a DEM using computer software routines.
  - The ASPRS vertical standard also uses the RMSE statistic, but only for well-defined features between contours containing interpretative elevations, or spot elevation points.
  - The limiting RMSE for Class 1 contours is one-third of the contour interval. Testing for vertical map compliance is also performed by independent, equal, or higher accuracy ground survey methods, such as differential leveling.
  - Table 4 summarizes the limiting vertical RMSE for well-defined points, as checked by independent surveys at the full (ground) scale of the map.

- d. Map accuracy quality assurance testing and certification.
  - Independent map testing is a quality assurance function that is performed independent of normal quality control during the mapping process.
  - Specifications and/or contract provisions should indicate the requirement (or option) to perform independent map testing.
  - Independent map testing is rarely performed for engineering and construction surveys.
  - If performed, map testing should be completed within a fixed time period after delivery, and if performed by contract, after proper notification to the contractor.
  - In accordance with the ASPRS standard, the horizontal and vertical accuracy of a map is checked by comparing measured coordinates or elevations from the map (at its intended target scale) with spatial values determined by a check survey of higher accuracy.

- The check survey should be at least twice (preferably three times) as accurate as the map feature tolerance given in the ASPRS tables, and a minimum of 20 points tested.
- Maps and related geospatial databases found to comply with a particular ASPRS standard should have a statement indicating that standard.
- The compliance statement should refer to the data of lowest accuracy depicted on the map, or, in some instances, to specific data layers or levels.
- The statement should clearly indicate the target map scale at which the map or feature layer was developed.
- When independent testing is not performed, the compliance statement should clearly indicate that the procedural mapping specifications were designed and performed to meet a certain ASPRS map classification, but that a rigid compliance test was not performed.
- Published maps and geospatial databases whose errors exceed those given in a standard should indicate in their legends or metadata files that the map is not controlled and that dimensions are not to scale.
- This accuracy statement requirement is especially applicable to GIS databases that may be compiled from a variety of sources containing known or unknown accuracy reliability.

- e. National Standard for Spatial Data Accuracy (NSSDA).
  - The traditional small-scale "United States National Map Accuracy Standard" (Bureau of the Budget 1947) has been revised by the FGDC as the NSSDA ("Geospatial Positioning Accuracy Standards, PART 3: National Standard for Spatial Data Accuracy").
  - This latest version of the NSSDA indicates it is directly based on the ASPRS standard; however, the ASPRS coordinate-based standard is converted to a 95% radial error statistic and the vertical standard is likewise converted from a one-sigma (68%) to 95% standard.
  - The NSSDA defines positional accuracy of spatial data, in both digital and graphic form, as derived from sources such as aerial photographs, satellite imagery, or other maps.
  - Its purpose is to facilitate the identification and application of spatial data by implementing a well-defined statistic (i.e. the 95% confidence level) and testing methodology.

- As in the ASPRS standard, accuracy is assessed by comparing the positions of well-defined data points with positions determined by higher accuracy methods, such as ground surveys.
- Unlike the above ASPRS tables, the draft NSSDA standard does not define pass-fail criteria--data and map producers must determine what accuracy exists for their data.
- Users of that data determine what constitutes acceptable accuracies for their applications.
- Unlike the ASPRS standard that uses the RMSE statistic in the X, Y, and Z planes, the NSSDA defines horizontal spatial accuracy by circular error of a data set's horizontal (X & Y) coordinates at the 95% confidence level.
- Vertical spatial data is defined by linear error of a data set's vertical (Z) coordinates at the 95% confidence level.

- ASPRS lineal horizontal accuracies in X and Y can be converted to NSSDA radial accuracy by multiplying the limiting RMSE values by 2.447, that is:

$$Radial\ Accuracy_{\ NSSDA} = 2.447 \cdot RMSE_{\ ASPRS\text{--}\ X\ or\ Y}$$

- ASPRS 1-sigma (68%) vertical accuracies can be converted to NSSDA 95% lineal accuracy by multiplying the limiting RMSE values by 1.96, or:

$$Vertical\ Accuracy_{\ NSSDA} = 1.96 \cdot RMSE_{\ ASPRS\text{ -- }Z}$$

- In time, it is expected that the NSSDA will be the recognized standard for specifying the accuracy of all mapping and spatial data products, and the ASPRS standard will be modified to 95% confidence level specifications.

• f. Other mapping standards.
  - When work is performed for DOD tactical elements or other Federal agencies or overseas, mapping standards other than ASPRS may be required.

Photogrammetric Mapping Standards and Specifications

- Most smaller scale (e.g., less than 1 inch = 100 ft or 1:1,200) engineering topographic mapping and GIS data base development is accomplished by aerial mapping techniques.
- The ASPRS standards should be used in specifying photogrammetric mapping accuracy requirements.
- Procedures for developing photogrammetric mapping specifications are contained in EM 1110-1-1000 (Photogrammetric Mapping).
- This manual contains guidance on specifying flight altitudes, determining target scales, and photogrammetric mapping cost estimating techniques.
- A full contract guide specification is also contained in an appendix to EM 1110-1-1000.

Cadastral or Real Property Survey Accuracy Standards

- a. General.
  - Many State codes, rules, statutes, or general professional practices prescribe minimum technical standards for real property surveys.
  - Corps in-house surveyors or contractors should follow applicable State technical standards for real property surveys involving the determination of the perimeters of a parcel or tract of land by establishing or reestablishing corners, monuments, and boundary lines, for the purpose of describing, locating fixed improvements, or platting or dividing parcels.
  - Although some State standards relate primarily to accuracies of land and boundary surveys, other types of survey work may also be covered in some areas.
  - Refer to ER 405-1-12, (Real Estate Handbook), and the "Manual of Instructions for the Survey of the Public Lands of the United States" (US Bureau of Land Management 1973) for additional technical guidance on performing cadastral surveys, or surveys of private lands abutting or adjoining Government lands.

- b. ALTA/ACSM standards.
  - Real property survey accuracy standards recommended by ALTA/ACSM are contained in "Minimum Standard Detail Requirements for ALTA/ACSM Land Title Surveys" (ALTA 1999), a portion of which is excerpted below. (Note that these ALTA standards are periodically updated).
  - This standard was developed to provide a consistent national standard for land title surveys and may be used as a guide in specifying accuracy closure requirements for USACE real property surveys.
  - However, it should be noted that the ALTA/ACSM standard itself not only prescribes closure accuracies for land use classifications but also addresses specific needs particular to land title insurance matters.
  - The standards contain requirements for detailed information and certification pertaining to land title insurance, including information discoverable from the survey and inspection that may not be evidenced by the public records.
  - The standard also contains a table as to optional survey responsibilities and specifications that the title insurer may require.
  - USACE cadastral surveys not involving title insurance should follow State minimum standards, not ALTA/ACSM standards.
  - On land acquisition surveys which may require title insurance, the decision to perform an ALTA/ACSM standard survey, including all optional survey responsibilities and specifications, should come from the project sponsor.
  - Meeting ALTA/ACSM Urban Class accuracy standards is considered impractical for small tracts or parcels less than 1 acre in size.

Accuracy Standards for ALTA-ACSM Land Title Surveys

- *Introduction*
  - *These Accuracy Standards address Positional Uncertainty and Minimum Angle, Distance and Closure Requirements for ALTA-ACSM Land Title Surveys.*
  - *In order to meet these standards, the Surveyor must assure that the Positional Uncertainties resulting from the survey measurements made on the survey do not exceed the allowable Positional Tolerance.*
  - *If the size or configuration of the property to be surveyed or the relief, vegetation, or improvements on the property will result in survey measurements for which the Positional Uncertainty will exceed the allowable Positional Tolerance, the surveyor must alternatively apply the within table of "Minimum Angle, Distance and Closure Requirements for Survey Measurements Which Control Land Boundaries for ALTA-ACSM Land Title Surveys" to the measurements made on the survey or employ, in his or her judgment, proper field procedures, instrumentation and adequate survey personnel in order to achieve comparable results.*

- *The lines and corners on any property survey have uncertainty in location which is the result of (1) availability and condition of reference monuments, (2) occupation or possession lines as they may differ from record lines, (3) clarity or ambiguity of the record descriptions or plats of the surveyed tracts and its adjoiners and (4) Positional Uncertainty.*
- *The first three sources of uncertainty must be weighed as evidence in the determination of where, in the professional surveyor's opinion, the boundary lines and corners should be placed.*
- *Positional Uncertainty is related to how accurately the surveyor is able to monument or report those positions.*
- *Of these four sources of uncertainty, only Positional Uncertainty is controllable, although due to the inherent error in any measurement, it cannot be eliminated.*
- *The first three can be estimated based on evidence; Positional Uncertainty can be estimated using statistical means.*

- *The surveyor should, to the extent necessary to achieve the standards contained herein, compensate or correct for systematic errors, including those associated with instrument calibration.*
- *The surveyor shall use appropriate error propagation and other measurement design theory to select the proper instruments, field procedures, geometric layouts and computational procedures to control and adjust random errors in order to achieve the allowable Positional Tolerance or required traverse closure.*
- *If radial survey methods are used to locate or establish points on the survey, the surveyor shall apply appropriate procedures in order to assure that the allowable Positional Tolerance of such points is not exceeded.*

- *Definitions:*
  - *"Positional Uncertainty" is the uncertainty in location, due to random errors in measurement, of any physical point on a property survey, based on the 95% confidence level.*
  - *"Positional Tolerance" is the maximum acceptable amount of Positional Uncertainty for any physical point on a property survey relative to any other physical point on the survey, including lead-in courses.*
  - *Computation of Positional Uncertainty*
    - *The Positional Uncertainty of any physical point on a survey, whether the location of that point was established using GPS or conventional surveying methods, may be computed using a minimally constrained, correctly weighted least squares adjustment of the points on the survey.*

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- *Positional Tolerances for Classes of Survey*
  - *0.07 feet (or 20mm) + 50ppm*
- *Application of Minimum Angle, Distance, and Closure Requirements*
  - *The combined precision of a survey can be statistically assured by dictating a combination of survey closure and specified procedures for an ALTA/ACSM Land Title Survey.*
  - *ACSM, NSPS and ALTA have adopted the following specific procedures in order to assure the combined precision of an ALTA/ACSM Land Title Survey.*
  - *The statistical base for these specifications is on file at ACSM and available for inspection.*

Hydrographic Surveying Accuracy Standards

- Hydrographic surveys are performed for a variety of engineering, construction, and dredging applications in USACE.
- Accuracy standards, procedural specifications, and related technical guidance are contained in EM 1110-2-1003 (Hydrographic Surveying).
- This manual should be attached to any A-E contract containing hydrographic surveying work, and must be referenced in construction dredging contracts involving in-place measurement and payment.
- Standards in this manual apply to Corps river and harbor navigation project surveys, such as dredge measurement and payment surveys, channel condition surveys of inland and coastal Federal navigation projects, beach renourishment surveys, and surveys of other types of marine structures.
- Accuracy standards are given for different project conditions and depths.
- Standards for nautical charting surveys or deep-water bathymetric charting surveys should conform to applicable DOD, National Ocean Survey (NOS), or US Naval Oceanographic Office (USNAVOCEANO) accuracy and chart symbolization criteria.

Structural Deformation Survey Standards

- Deformation monitoring surveys of Corps structures require high line vector and/or positional accuracies to monitor the relative movement of monoliths, walls, embankments, etc.
- Deformation monitoring survey accuracy standards vary with the type of construction, structural stability, failure probability and impact, etc.
- Since many periodic surveys are intended to measure "long-term" (e.g., monthly or yearly changes) deformations relative to a stable network, lesser survey precisions are required than those needed for short-term structural deflection type measurements.
- Long-term structural movements measured from points external to the structure may be tabulated or plotted in either X-Y-Z or by single vector movement normal to a potential failure plane.
- Accuracy standards and procedures for structural deformation surveys are contained in EM 1110-2-1009 (Structural Deformation Surveying).
- Horizontal and vertical deformation monitoring survey procedures are performed relative to a control network established for the structure.
- Ties to the National Spatial Reference System are not necessary other than for general reference, and then need only USACE Third-Order connection.

Geodetic Control Survey Standards

- Geodetic control surveys are usually performed for the purpose of establishing a basic framework of the National Spatial Reference System (NSRS).
- These geodetic network densification survey functions are clearly distinct from the traditional engineering and construction surveying and mapping standards covered in this chapter.
- Geodetic control surveys of permanently monumented control points that are incorporated in the NSRS must be performed to far more rigorous standards and specifications than are control surveys used for general engineering, construction, mapping, or cadastral purposes.
- When a project requires NSRS densification, or such densification is a desirable by-product and is economically justified, USACE Commands should conform to published FGDC survey standards and specifications.
- This includes related automated data recording, submittal, project review, and adjustment requirements mandated by FGDC and the National Geodetic Survey.

- Geodetic survey accuracy and procedural specifications published by the FGDC or NGS include:
  - "Standards and Specifications for Geodetic Control Networks" (FGCS 1984)
  - "Input Formats and Specifications of the National Geodetic Survey Data Base," NOAA, National Geodetic Survey, (NOAA 1994)
  - "Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques (Preliminary)" (FGCS 1988)
  - "Guidelines for Submitting GPS Relative Positioning Data to the National Geodetic Survey" (NGS 1988)
  - "Geospatial Positioning Accuracy Standards--Part 2: Standards for Geodetic Networks" (FGDC 1998b)
  - "Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm)" (NOAA 1997)

- Copies of these specifications and standards can be downloaded from the NGS website.
- These FGCS/NGS standards and specifications should rarely be specified for Corps control surveys in that they prescribe far more demanding criteria than that needed to establish control for most engineering projects.
- These FGDC/NGS standards can also easily add 50% or more time and cost to a control survey project.
- Part 2 of the FGDC Geospatial Positioning Accuracy Standards (Standards for Geodetic Networks)-- FGDC 1988b-- prescribes a positional accuracy criteria instead of the traditional linear closure (misclosure) criteria.
- It is expected that this positional accuracy standard will gradually replace the misclosure standards in Tables 1 and 2.

- This new standard is excerpted in the table below.

Accuracy Classification	95-Percent Confidence Less Than or Equal to:
1-Millimeter	0.001 meters
2-Millimeter	0.002 "
5-Millimeter	0.005 "
1-Centimeter	0.010 "
2-Centimeter	0.020 "
5-Centimeter	0.050 "
1-Decimeter	0.100 "
2-Decimeter	0.200 "
5-Decimeter	0.500 "
1-Meter	1.000 "
2-Meter	2.000 "
5-Meter	5.000 "
10-Meter	10.000 "

NOTE: The classification standard for geodetic networks is based on accuracy. Accuracies are categorized separately according to horizontal, ellipsoid height, and orthometric height. Note: although the largest entry in the table is 10 meters, the accuracy standards can be expanded to larger numbers if needed.

Table 5: FGDC Part 2 Accuracy Standards for Geodetic Networks Horizontal, Ellipsoid Height, and Orthometric Height.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

State and Local Accuracy Standards

- Most State and local governments prescribe survey and map accuracy standards.
- These are usually similar to those standards given in the previous tables in this chapter.
- State surveyor licensing boards may prescribe "minimum technical standards" for various real property surveys.
- State transportation departments may have additional standards unique to their design and construction requirements.

- a. General state surveying and mapping standards.
  - Below is an excerpt of surveying accuracy standards taken from the Florida Administrative Code (FAC 2003).
  - These standards for general boundary surveys are representative of minimum technical standards used by many states.
  - *(1) Survey and Map Accuracy*
    - *(a) REGULATIONAL OBJECTIVE:*
      - *The public must be able to rely on the accuracy of measurements and maps produced by a surveyor and mapper.*
      - *In meeting this objective, surveyors and mappers must achieve the following minimum standards of accuracy, completeness, and quality.*

- *(b) The accuracy of the survey measurements shall be premised upon the type of survey and the expected use of the survey and map.*
  - *All measurements must be in accordance with the United States standard, using either feet or meters.*
  - *Records of these measurements shall be maintained for each survey by either the individual surveyor and mapper or the surveying and mapping business entity.*
  - *Measurement and computation records must be dated and must contain sufficient data to substantiate the survey map and insure that the accuracy portion of these standards has been met.*

- *(c) Vertical Control:*
  - *Field-measured control for elevation information shown upon survey maps shall be based on a level loop.*
  - *Closure in feet must be accurate to a standard of plus or minus .05 ft. times the square root of the distance in miles.*
  - *All surveys and maps with elevation data shall indicate the datum and a description of the benchmark(s) upon which the survey is based.*
  - *Minor elevation data may be obtained on an assumed datum provided the base elevation of the datum is obviously different than the established datum.*

- *(d) Vertical Feature Accuracy:*
  - *1. If contour lines are shown, then sufficient data must be obtained in order to insure that 90% of ground point elevations taken from contours are within 1/2 of the contour interval, and the remainder are not in error more than the contour interval.*
  - *2. For surveys performed by photogrammetric methods, vertical positional accuracy of map elevations, contours, or other forms of terrain models must be stated.*
    - ✓ *The stated accuracy is a plus or minus tolerance that encompasses 90% of elevation differences between survey measured values and ground truth.*
    - ✓ *All such survey maps or reports with elevation data shall have a statement to the effect: "Elevations of well-identified features contained in this survey been measured to an estimated vertical positional accuracy of: ____ (ft) (m)."*
    - ✓ *If different accuracy levels exist for different features, then applicable features and accuracies shall be identified with similar statements.*

- *(e) Horizontal Control:*
  - *All surveys and maps expressing or displaying features in coordinate position shall indicate the coordinate datum and a description of the control points upon which the survey is based.*
  - *Minor coordinate data may be obtained on an assumed datum provided the numerical basis of the datum is obviously different than an established datum.*
  - *The accuracy of field-measured control measurements shall be statistically verified by measurement and calculation of a closed geometric figure.*
  - *All control measurements shall be made with a transit and steel tape, or devices with equivalent or higher degrees of accuracy.*
  - *The relative distance accuracy must be better than the following:*
    - ✓ *Commercial/High Risk Linear: 1 foot in 10,000 feet;*
    - ✓ *Suburban: Linear: 1 foot in 7,500 feet;*
    - ✓ *Rural: Linear: 1 foot in 5,000 feet;*

- *(f) Horizontal Feature Accuracy (for surveys by photogrammetric methods only):*
  - *A survey and map's horizontal positional accuracy must be stated.*
  - *The stated accuracy is a plus or minus tolerance that encompasses 90% of coordinate differences between survey measured values and ground truth.*
  - *All survey maps or reports shall have a statement of the effect: "Well-identified features in this survey and map have been measured to an estimated horizontal positional accuracy of *[____] (ft) (m)."*
  - *If different accuracy levels exist for different features, then applicable features and accuracies shall be identified with similar statements.*

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- *(g) Map Plotting Accuracy:*
  - *The horizontal position of physical features surveyed by field methods must be plotted to within 1/20 of an inch at the map scale.*
- *(h) Intended Display Scale:*
  - *At the maximum intended display scale, a survey and map's positional accuracy value occupies 1/20" on the display.*
  - *All maps or reports of surveys produced by photogrammetric methods and delivered with digital coordinate files must contain a statement to the effect of: "This map is intended to be displayed at a scale of 1/ *[______] or smaller."*

- (2) Other Provisions that Apply to All Surveys and Maps.
  - (a) REGULATIONAL OBJECTIVE:
    - In order to avoid misuse of a survey and map, the surveyor and mapper must adequately communicate the survey results to the public through a map, report, or report with an attached map.
    - Any survey map or report must identify the responsible surveyor and mapper and contain standard content.
    - In meeting this objective, surveyors and mappers must meet the following minimum standards of accuracy, completeness, and quality:

- (b) Each survey map and report shall state the type of survey it depicts consistent with the types of surveys defined in Rule 61G17-6.002(8)(a)-(k), F.A.C.
  - The purpose of a survey, as set out in Rule 61G17- 6.002(8)(a)-(l), F.A.C., dictates the type of survey to be performed and depicted, and a licensee may not avoid the minimum standards required by rule of a particular survey type merely by changing the name of the survey type to conform with what standards or lack of them the licensee chooses to follow.

- *(c) All survey maps and reports must bear the name, certificate of authorization number, and street and mailing address of the business entity issuing the map and report, along with the name and license number of the surveyor and mapper in responsible charge.*
  - *The name, license number, and street and mailing address of a surveyor and mapper practicing independent of any business entity must be shown on each survey map and report.*

- *(d) All survey maps must reflect a survey date, which is the date of the field survey or the date of image acquisition for photogrammetric surveys.*
  - *If the graphics of a map are revised, but the survey date stays the same, the map must list dates for all revisions.*
- *(l) Responsibility Clearly Stated.*
  - *The responsibility for all mapped features must be clearly depicted on any map or report signed by a Florida licensed surveyor and mapper.*
  - *In the case that features surveyed by the signing surveyor and mapper have been integrated with features surveyed by others, then the full extent of responsibility shall be clearly depicted on the map or report, and the signing surveyor and mapper shall include in the map or report an assessment of the quality and accuracy of all mapped features delivered.*

- b. DOT control survey standards.
  - The following Third-Order survey standards shown in the following two figures are from the CALTRANS Surveys Manual.

Specifications	Traverse/Network Resection Double Tie
Check vertical index error	Daily
Check horizontal collimation	Daily
Measure instrument height and target height	Begin and end of each setup
Use plummet to check position of target and instrument over points	Begin and end of each setup
Measure temperature and pressure and enter ppm correction into total station	First set-up of day
Measure distance to backsight and foresight at each setup	Required
Observe traverse multiple ties to improve least squares adjustment	As Feasible
Close all traverses	Required
Horizontal angle observations, minimum	3D, 3R
Vertical angle observations, minimum	3D, 3R
Angular rejection limit, residual not to exceed	5"
Maximum value for the standard error of the mean	1.2"
Minimum distance measurement to meet horizontal accuracy standard	50 m
Minimum number of distance measurements	3
Distance rejection limit: residual not to exceed	2mm + 2 ppm
Maximum distance measurement to meet vertical accuracy standard	100 m

Figure 16: CALTRANS Third-Order horizontal control standards (Total Station).
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

Operation/Specification	Compensator-Level Three-Wire Observation	Compensator-Level Single-Wire Observation	Electronic/Digital Bar Code Level
Difference in length between fore and back sights, not to exceed per setup	10 m	10 m	10 m
Cumulative difference in length between fore and backsights, not to exceed per loop or section	10 m	10 m	10 m
Maximum sight lengths	90 m	90 m	90 m (See Note 1)
Minimum ground clearance of sight line	0.5 m	0.5 m	0.5 m
Maximum section misclosure	$12 \text{ mm} \times \sqrt{D}$ (See Note 2)	$12 \text{ mm} \times \sqrt{D}$ (See Note 2)	$8 \text{ mm} \times \sqrt{D}$ (See Note 2)
Maximum loop misclosure	$12 \text{ mm} \times \sqrt{E}$ (See Note 3)	$12 \text{ mm} \times \sqrt{E}$ (See Note 3)	$8 \text{ mm} \times \sqrt{E}$ (See Note 3)
Difference between top and bottom interval not to exceed	0.30 of rod unit	N/A	N/A
Collimation (Two-Peg) Test	Daily (See Note 4) (not to exceed 2 mm)	Daily	Daily
Minimum number of readings (Use repeat measure option for each observation)	N/A	N/A	3 (See Note 5)

Figure 17: CALTRANS Third-Order differential leveling standards.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- The first standard is for establishing permanent Third-Order horizontal control using a total station.
- The second is a Third-Order standard for differential leveling--covering different types of levels.
- The figure on the following slide depicts a CALTRANS accuracy standard for setting primary control around a project site.

CALTRANS ORDER (Note 1)	STANDARDS			MONUMENT SPACING AND SURVEY METHODS (Note 2)			APPLICATION – TYPICAL SURVEYS	
	CLASSICAL		POSITIONAL	MONUMENT SPACING (MINIMUM)	TYPICAL SURVEY METHOD			
	HORIZONTAL (Note 4)	VERTICAL (Note 5)			HORIZONTAL	VERTICAL	HORIZONTAL	VERTICAL
B (Note 3)	1:1,000,000	Not Applicable	Per NGS Specifications	10 k	GPS: Static	Not Applicable	High Precision Geodetic Network (HPGN)	Not Applicable
First (Note 3)	1:100,000 (Note 10)	$e = 5\sqrt{E}$	Per NGS Specifications	3 k	GPS: Static Fast Static	Bar Code	Basic (Corridor) Control – HPGN-D Project Control – Horizontal (perferred, when feasible)	Rarely used. Crustal Motion Surveys, etc.
Second	1:20,000	$e = 8\sqrt{E}$	(Note 8)	500 m	GPS: Static Fast Static TSSS: Net Traverse	Bar Code 3-Wire TSSS: Trig	Project Control – Horizontal (see First Order also)	Basic (Corridor) Control HPGN and HPGN-D Project Control
Third	1:10,000	$e = 12\sqrt{E}$	(Note 8)	As Required	GPS: Static Fast Static Kinematic RTK (Note 13) TSSS: Net Traverse Resection Double Tie (Note 9)	Bar Code Single Wire TSSS: Trig GPS: Static (Note 7) RTK (Note 11)	Supplemental Control > Engineering > Construction • Interchange • Major Structure Photo. Control – Horizontal Right of Way Surveys Construction Surveys (Note 6) Topographic Surveys (Note 6) Major Structure Points (Staked)	Project Control – Vertical Supplemental Control Photo. Control – Vertical Construction Surveys (Note 6) Topographic Surveys (Note 6) Major Structure Points (Staked)
G (General)	As required, see appropriate survey procedure section in this manual for accuracy standards/tolerances.			Not Applicable	GPS: Fast Static Kinematic RTK TSSS: Radial	GPS: Fast Static Kinematic, RTK (Note 12) TSSS: Trig Single Wire Direct Elevation Rod	Topographic Surveys (Data Points) Supplement Design Data Surveys Construction Surveys (Staked Points) Environmental Surveys GIS Data Surveys Right of Way Flagging	

Notes
1. The standards, specifications, and procedures included in this Manual are based on Federal Geodetic Control Subcommittee (FGCS) standards and specifications. Except where otherwise noted, the FGCS requirements have been modified to meet Caltrans needs.
2. Refer to other Manual sections for detailed procedural specifications for specific survey methods and types of surveys.
3. "B" Order and First Order surveys are performed to FGCS standards and specifications or other requirements approved by National Geodetic Survey
4. Distance accuracy standard.
5. Closure between established control; e = maximum misclosure in mm, E = distance in km.
6. Survey setup points used for radial stake out.
7. For example a static GPS may be used to establish NAVD88 at the project site from a distant NAVD National Spatial Reference System Control.
8. As required by the local survey needs.
9. Instead of including a point as a network point, certain survey points may be positioned by observations from two or more control points (i.e., double tied). If survey points are not included in a network, double ties must be performed to ensure that blunders are eliminated and the positions established are within stated accuracy standard. Double tie procedures should be only used when appropriate; possible examples are photo control points, landnet and monumentation points, and major structure stake points.
10. The distance accuracy standard for Basic (Corridor) Control – HPGN-D surveys is 1:500,000.
11. Not to include vertical project control or vertical for major structure points.
12. Not to include pavement elevations.
13. Not to include major structures.

Figure 18: CALTRANS accuracy classifications and standards.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- The classifications and closure standards are identical with those in Tables 1 and 2.
- The "G" classification is roughly comparable to USACE "4th Order" classification

CADD/GIS Technology Center Standards

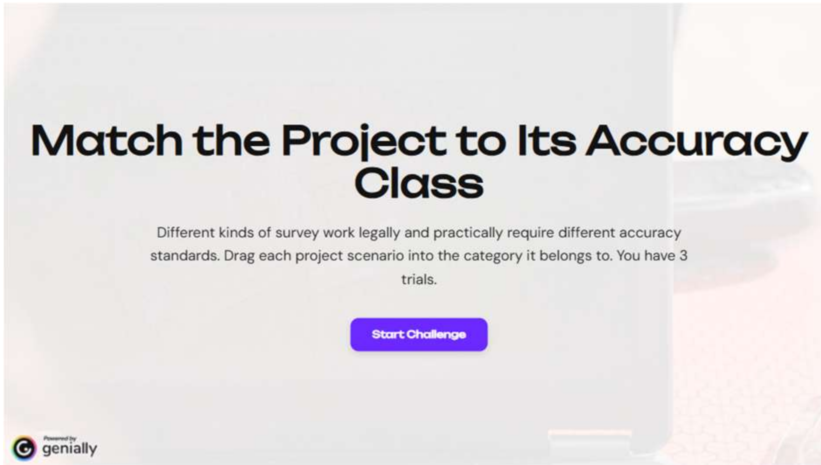
- The CADD/GIS Technology Center [for Facilities, Infrastructure, and Environment] is located at the USACE Waterways Experiment Station in Vicksburg, MS.
- The Center's primary mission is to serve as a multi-service vehicle to set computer-aided design and drafting (CADD) and geographic information system (GIS) standards; coordinate CADD/GIS facilities systems within the Department of Defense (DOD); promote CADD/GIS system integration; support centralized CADD/GIS hardware and software acquisition; and provide assistance for the installation, training, operation, and maintenance of CADD and GIS systems.
- The intent of the CADD/GIS Technology Center standards development initiatives has been to develop usable CADD, GIS, and facility management (FM) standards that will satisfy the project life-cycle concept for digital data.
- This concept requires a set of CADD, GIS, and FM standards for initial data collection, analysis, design, construction, and subsequent master planning, facility management, and maintenance.
- This allows for direct integration from CADD engineering design or as-builts to such GIS analysis tasks as master planning and FM.

- The Center has issued a number of geospatial standards and related CADD/GIS guidance.
- Some of these standards include:
  - Spatial Data Standard for Facilities, Infrastructure, and Environment (SDSFIE)
  - Facility Management Standard for Facilities, Infrastructure, and Environment (FMSFIE)
  - A/E/C CADD Standard These A/E/C CADD standards define symbology, level/layer assignments, drafting templates, sheet layouts, and other criteria required in a CADD environment.
- The SDSFIE Standards define the attributes and attribute values for geospatial data features.
- These standards should be specified for in-house or A-E services requiring delivery of CADD, GIS, and other spatial and geospatial data covered by this chapter.

Mandatory Standards

- The accuracy standards in the following tables in this chapter are considered mandatory.
  - Table 1: Minimum Closure Accuracy Standards for Engineering and Construction Surveys
  - Table 2: Minimum Elevation Closure Accuracy Standards for Engineering and Construction Surveys
  - Table 3a: ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Meters) for Well-Defined Points
  - Table 3b: ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Feet) for Well-Defined Points
  - Table 4a: ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Meters)
  - Table 4b: ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Feet)
- HQUSACE has directed that geospatial data collected for architectural, engineering, and construction projects shall be compliant with the A/E/C CADD Standard and/or the SDSFIE Standard.
- This includes data that is collected, developed, or contracted for, and/or otherwise executed by the Corps of Engineers.
- Topographic survey data falls within this directive.

Check the next slide to explore an interactive GIF about a Sorting Challenge



GEODETIC REFERENCE DATUMS AND LOCAL COORDINATE SYSTEMS

Purpose and Background

- This chapter provides guidance on geodetic reference datums, coordinate systems, and local horizontal and vertical reference systems that are used to georeference Corps military construction and civil works projects.
- Use of State Plane Coordinate Systems (SPCS) is covered in detail since these systems are most commonly used to reference topographic surveys of local projects.
- Transformations between datums and coordinate systems are discussed.

- a. Topographic surveys can be performed on any coordinate system.
  - Most localized total station topographic surveys are initiated on (or referenced to) an arbitrary coordinate grid system, e.g., X=5,000 ft, Y=5,000 ft, Z=100 ft, and often elevation or scale reductions are ignored.
  - Planimetric and topographic data points collected on this arbitrary grid in a data collector are then later translated, rotated, scaled, and/or "best fit" to some established geographical reference system--e.g., the local State Plane Coordinate System (SPCS).

- b. The process of converting the observed topographic points on the arbitrary grid system to an established geographical reference system (e.g., SPCS) is termed a "datum transformation."
  - In order to perform this transformation, a few points (preferably three or more) in the topographic database must be referenced to the external reference system.
  - These "control" points on a topographic survey have been previously established relative to an installation or project's primary control network.
  - They normally were established using more accurate "geodetic control" survey procedures, such as differential leveling, static or kinematic DGPS observations, or total station traverse.

- c. Most USACE topographic surveys require "control surveys" to bring in a geodetic reference network to the local project site where detailed topographic surveys are performed.
  - It is important that the correct geodetic reference network is used, and that it is consistent with the overall installation or project reference system.
  - It may also important that these reference systems conform to regional or nationwide reference systems, such as the National Spatial Reference System (NSRS), North American Datum of 1983 (NAD 83), or the North American Vertical Datum of 1988 (NAVD 88).
  - These various reference datums and systems are discussed in this chapter.

- d. Not all topographic surveys require a rigid reference to some local or regional geographic coordinate system, and thus do not need time consuming and expensive preliminary control surveys.
  - Some project feature or on going construction applications may only require a simple local reference--for example, a single monument with assumed or scaled coordinates and an arbitrary reference azimuth may suffice.

- e. Other topographic surveys outside Army installations or Corps civil project areas may require rigid references to established property boundaries (corner pins, section corners, road intersections/centerlines, etc.).
  - These ties to legal boundaries and corners will thus establish the reference system by which all topographic survey features are detailed.
  - Regional geodetic or SPCS networks may or may not be required on such surveys, depending on local practice or statute.
- f. For additional details on geodetic datums and coordinate systems, refer to EM 1110-1-1003 (NAVSTAR GPS Surveying).

SECTION I. Geodetic Reference Systems

General

- The discipline of surveying consists of locating points of interest on the surface of the earth.
- The positions of points of interest are defined by coordinate values that are referenced to a predefined mathematical surface.
- In geodetic surveying, this mathematical surface is called a datum, and the position of a point with respect to the datum is defined by its coordinates.
- The reference surface for a system of control points is specified by its position with respect to the earth and its size and shape.
- A datum is a coordinate surface used as reference figure for positioning control points.
- Control points are points with known relative positions tied together in a network.
- Densification of the network refers to adding more fixed control points to the network.
- Both horizontal and vertical datums are commonly used in surveying and mapping to reference coordinates of points in a network.

- Reference systems can be based on the geoid, ellipsoid, or a plane.
- The physical earth's gravity force can be modeled to create a positioning reference frame that rotates with the earth.
- The geoid is such a surface (an equipotential surface of the earth's gravity field) that best approximates Mean Sea Level (MSL).
- The orientation of this surface at a given point on geoid is defined by the plumb line.
- The plumb line is oriented tangent to the local gravity vector.
- Surveying instruments can be readily oriented with respect to the gravity field because its physical forces can be sensed with simple mechanical devices.
- Such a reference surface is developed from an ellipsoid of revolution that best approximates the geoid.
- An ellipsoid of revolution provides a well-defined mathematical surface to calculate geodetic distances, azimuths, and coordinates.

Geodetic Coordinates

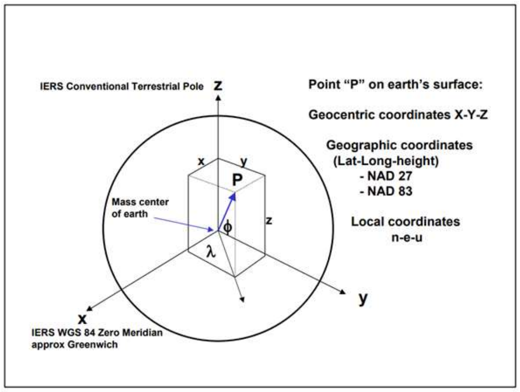


Figure 19: Earth-centered earth-fixed coordinate reference frames.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- A coordinate system is defined by the location of the origin, orientation of its axes, and the parameters (coordinate components) which define the position of a point within the coordinate system.
- Terrestrial coordinate systems are widely used to define the position of points on the terrain because they are fixed to the earth and rotate with it.
- The origin of terrestrial systems can be specified as either geocentric (origin at the center of the earth, such as NAD 83) or topocentric (origin at a point on the surface of the earth, such as NAD 27).
- The orientation of terrestrial coordinate systems is described with respect to its poles, planes, and axes.
- The primary pole (Z in the previous figure) is the axis of symmetry of the coordinate system, usually parallel to the rotation axis of the earth, and coincident with the semi-minor axis of the reference ellipsoid.
- The reference planes that are perpendicular to the primary polar axis are the equator (zero latitude) and the Greenwich meridian plane (zero longitude).
- Parameters for point positioning within a coordinate system refer to the coordinate components of the system (either Cartesian or curvilinear).

- a. Geocentric coordinates.
  - Geocentric coordinates have an origin at the center of the earth.
  - GPS coordinates are initially observed on this type of reference system.
  - For example, a coordinate on such a system might be displayed on a GPS receiver as:

```
X =    668400.506 m
Y =  -4929214.152 m
Z =   3978967.747 m
```

  - GPS receivers will transform these geocentric coordinates into a geographic coordinate system described below.

- b. Geodetic or Geographic coordinates.
  - Geographic coordinate components consist of:
    - latitude (φ),
    - longitude (λ),
    - ellipsoid height (h).
  - Geodetic latitude, longitude, and ellipsoid height define the position of a point on the surface of the Earth with respect to some "reference ellipsoid."
  - The most common reference ellipsoid used today is the WGS 84, which will be described in more detail in a later section.

- (1) Geodetic latitude (φ).
  - The geodetic latitude of a point is the acute angular distance between the equatorial plane and the normal through the point on the ellipsoid measured in the meridian plane.
  - Geodetic latitude is positive north of the equator and negative south of the equator.

- (2) Geodetic longitude (λ).
  - The geodetic longitude is the angle measured counter-clockwise (east), in the equatorial plane, starting from the prime meridian (Greenwich meridian), to the meridian of the defined point.
  - In the continental United States, longitude is commonly reported as a west longitude.
  - To convert easterly to westerly referenced longitudes, the easterly longitude must be subtracted from 360 deg.
  - East-West Longitude Conversion:

$$\lambda\,(W) \;=\; [\,360\; - \;\lambda\,(E)\,]$$

  - For example:

$$\lambda\,(E) = 282^{\,d}\,52^{\,m}\,36.345^{\,s}\,E$$

$$\lambda\,(W) = [\,360^{\,d} - 282^{\,d}\,52^{\,m}\,36.345^{\,s}\,E\,]$$
$$\lambda\,(W) = 77^{\,d}\,07^{\,m}\,23.655^{\,s}\,W$$

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- (3) Ellipsoid Height (h).
  - The ellipsoid height is the linear distance above the reference ellipsoid measured along the ellipsoidal normal to the point in question.
  - The ellipsoid height is positive if the reference ellipsoid is below the topographic surface and negative if the ellipsoid is above the topographic surface.
- (4) Geoid Separation (N).
  - The geoid separation (or often termed "geoidal height") is the distance between the reference ellipsoid surface and the geoid surface measured along the ellipsoid normal.
  - The geoid separation is positive if the geoid is above the ellipsoid and negative if the geoid is below the ellipsoid.
- (5) Orthometric Height (H).
  - The orthometric height is the vertical distance of a point above or below the geoid.

Datums

- A datum is a coordinate surface used as reference for positioning control points.
- Both horizontal and vertical datums are commonly used in surveying and mapping to reference coordinates of points in a network.
- a. Geodetic datum.
  - Five parameters are required to define an ellipsoid-based datum.
  - The major semi-axis (a) and flattening (f) define the size and shape of the reference ellipsoid; the latitude and longitude of an initial point; and a defined azimuth from the initial point define its orientation with respect to the earth.
  - The NAD 27 and NAD 83 systems are examples of horizontal geodetic datums.

- b. Horizontal datum.
  - A horizontal datum is defined by specifying (1) the 2D geometric surface (plane, ellipsoid, sphere) used in coordinate, distance, and directional calculations, (2) the initial reference point (origin), and (3)a defined orientation, azimuth or bearing from the initial point.
  - The “horizontal datum” for most topographic surveys is usually defined relative to the fixed control points (monuments and/or benchmarks) that were used to control the individual shots.
  - These “control points” may, in turn, be referenced to a local installation/compound control network and/or to a regional NSRS CORS station.

- c. Project datum.
  - A project datum is defined relative to local control and might not be directly referenced to a geodetic datum.
  - Project datums are usually defined by a system with perpendicular axes, and with arbitrary coordinates for the initial point, and with one (principal) axis oriented toward an assumed north.
  - A chainage-offset system may also be used as a reference, with the PIs (points of intersection) either marked points or referenced to some other coordinate system.

- d. Vertical datum.
  - A vertical datum is a reference system used for reporting elevations.
  - The two most common nationwide systems are the National Geodetic Vertical Datum of 1929 (NGVD 29) and the North American Vertical Datum of 1988 (NAVD 88).
  - Vertical elevations used on navigation, flood control, and hydropower projects may also be referenced to a variety of datums, such as:
    - Mean Sea Level (MSL)
    - Mean Low Water (MLW)
    - Mean Lower Low Water (MLLW)
    - Mean High Water (MHW)
    - International Great Lakes Datum (IGLD)
    - Low Water Reference Plane (LWRP)
    - Flat Pool Stage
    - Local Pool or Reservoir Capacity Reference Point

- Mean Sea Level (MSL) based elevations are used for most construction and topographic surveys--in particular those involving flood control or shoreline improvement/protection.
- It should be noted that MSL elevations are not the same as NGVD 29; and that MSL and NGVD 29 elevations can widely differ from NAVD 88 elevations--as much as 3 ft in western CONUS.
- MLLW elevations are used in referencing coastal navigation projects.
- MHW elevations are used in construction projects involving bridges and other crossings over navigable waterways.

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- e. The National Spatial Reference System (NSRS).
  - The NSRS is that component of the National Spatial Data Infrastructure (NSDI) which contains all geodetic control contained in the National Geodetic Survey (NGS) database.
  - This includes: A, B, First, Second and Third-Order horizontal and vertical control, geoid models, precise GPS orbits and Continuously Operating Reference Stations (CORS), and the National Shoreline as observed by NGS as well as data submitted by other Federal, State, and local agencies, academic institutions, and the private sector.

WGS 84 Reference Ellipsoid

- The WGS 84 ellipsoid is used to reference GPS satellite observations and is used to reduce observations onto the NAD 83 system.
- The origin of the WGS 84 Cartesian system is the earth's center of mass.
- The Z-axis is parallel to the direction of the Conventional Terrestrial Pole (CTP) for polar motion, as defined by the Bureau International Heure (BIH), and equal to the rotation axis of the WGS 84 ellipsoid.
- The X-axis is the intersection of the WGS 84 reference meridian plane and the CTP's equator, the reference meridian being parallel to the zero meridian defined by the BIH and equal to the X-axis of the WGS 84 ellipsoid.
- The Y-axis completes a right-handed, earth-centered, earth-fixed orthogonal coordinate system, measured in the plane of the CTP equator 90 degrees east of the X-axis and equal to the Y-axis of the WGS 84 ellipsoid.
- The DOD continuously monitors the origin, scale, and orientation of the WGS 84 reference frame and references satellite orbit coordinates to this frame.
- Updates are shown as WGS 84 (GXXX), where "XXX" refers to a GPS week number starting on 29 September 1996.

- Prior to development of WGS 84, there were several reference ellipsoids and interrelated coordinate systems (datums) that were used by the surveying and mapping community.
- The table on the following slide lists just a few of these reference systems along with their mathematical defining parameters.
- Note that GRS 80 is the actual reference ellipsoid for NAD 83; however, the difference between GRS-80 and WGS 84 ellipsoids is insignificant.
- Transformation techniques are used to convert between different datums and coordinate systems.
- Most GPS software has built in transformation algorithms for the more common datums.

Reference Ellipsoid	Coordinate System (Datum/Frame)	Semimajor axis (meters)	Shape (1/flattening)
Clarke 1866	NAD 27	6378206.4	1/294.9786982
WGS 72	WGS 72	6378135	1/298.26
GRS 80	NAD 83 (XX)	6378137	1/298.257222101
WGS 84	WGS 84 (GXXX)	6378137	1/298.257223563
ITRS	ITRF (XX)	6378136.49	1/298.25645

Table 6: Reference Ellipsoids and Related Coordinate Systems.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

Horizontal Datums and Reference Frames

- The following paragraphs briefly describe the most common datums used to reference CONUS projects.
- a. North American Datum of 1927 (NAD 27).
  - NAD 27 is a horizontal datum based on a comprehensive adjustment of a national network of traverse and triangulation stations.
  - NAD 27 is a best fit for the continental United States.
  - The fixed datum reference point is located at Meades Ranch, Kansas.
  - The longitude origin of NAD 27 is the Greenwich Meridian with a south azimuth orientation. The original network adjustment used 25,000 stations.
  - The relative precision between initial point monuments of NAD 27 is by definition 1:100,000, but coordinates on any given monument in the network contain errors of varying degrees.
  - As a result, relative accuracies between points on NAD 27 may be far less than the nominal 1:100,000.
  - The reference units for NAD 27 are US Survey Feet.
  - This datum is no longer supported by NGS, and USACE commands are gradually transforming their project coordinates over to the NAD 83 described below.
  - Approximate conversions of points on NAD 27 to NAD 83 may be performed using CORPSCON, a transformation program developed by ERDC/TEC.
  - Since NAD 27 contains errors approaching 10 m, transforming highly accurate GPS observations to this antiquated reference system is not the best approach.

- b. North American Datum of 1983 (NAD 83).
  - The nationwide horizontal reference network was redefined in 1983 and readjusted in 1986 by the National Geodetic Survey.
  - It is known as the North American Datum of 1983, adjustment of 1986, and is referred to as NAD 83 (1986).
  - NAD 83 used far more stations (250,000) and observations than NAD 27, including a few satellite-derived coordinates, to readjust the national network.
  - The longitude origin of NAD 83 is the Greenwich Meridian with a north azimuth orientation.
  - The fixed adjustment of NAD 83 (1986) has an average precision of 1:300,000.
  - NAD 83 is based upon the Geodetic Reference System of 1980 (GRS 80), an earth-centered reference ellipsoid which for most (but not all) practical purposes is equivalent to WGS 84.
  - With increasingly more accurate uses of GPS, the errors and misalignments in NAD 83 (1986) became more obvious (they approached 1 meter), and subsequent refinements outlined below have been made to correct these inconsistencies.

- c. High Accuracy Reference Networks (HARN).
  - Within a few years after 1986, more refined GPS measurements had allowed geodesists to locate the earth's center of mass with a precision of a few centimeters.
  - In doing so, these technologies revealed that the center of mass that was adopted for NAD 83 (1986) is displaced by about 2 m from the true geocenter.
  - These discrepancies caused significant concern as the use of highly accurate GPS measurements proliferated.
  - Starting with Tennessee in 1989, each state--in collaboration with NGS and various other institutions--used GPS technology to establish regional reference frames that were to be consistent with NAD 83.
  - The corresponding networks of GPS control points were originally called High Precision Geodetic Networks (HPGN).
  - Currently, they are referred to as High Accuracy Reference Networks (HARN).
  - This latter name reflects the fact that relative accuracies among HARN control points are better than 1 ppm, whereas relative accuracies among pre-existing control points were nominally only 10 ppm.
  - Positional differences between NAD 83 (1986) and NAD 83 (HARN) can approach 1 meter.

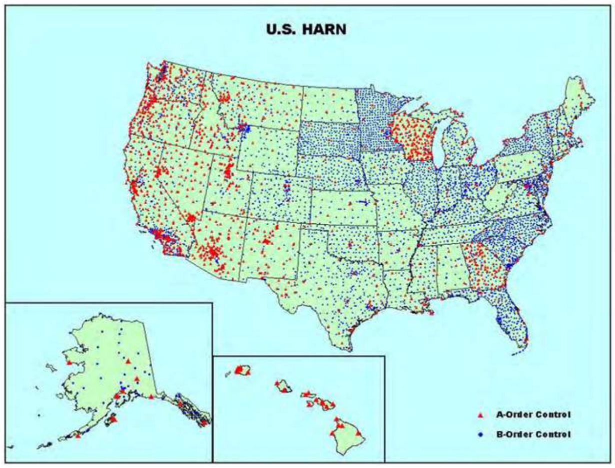


Figure 20: High Accuracy Reference Network control points.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- d. Continuously Operating Reference Stations (CORS).
  - The regional HARNs were subsequently further refined (or "realized") by NGS into a network of Continuously Operating Reference Stations, or CORS.
  - This CORS network was additionally incorporated with the International Terrestrial Reference System (ITRS), i.e. the ITRF.
  - CORS are located at fixed points throughout CONUS and at some OCONUS points.
  - This network of high-accuracy points can provide GPS users with centimeter level accuracy where adequate CORS coverage exists.
  - Coordinates of CORS stations are designated by the year of the reference frame, e.g., NAD 83 (CORS 96).
  - Positional differences between NAD 83 (HARN) and NAD 83 (CORS) are less than 10 cm.
  - More importantly, positional difference between two NAD 83 (CORSxx) points is typically less than 2 cm.
  - Thus, GPS connections to CORS stations will be of the highest order of accuracy.
  - USACE commands can easily connect and adjust GPSobserved points directly with CORS stations using a number of methods, including the NGS on-line program OPUS (On-Line Positioning User Service).
  - CORS are particularly useful when precise control is required in a remote area, from which a topographic survey may be performed.
  - With only 1 to 2 hours of static DGPS observations, reference points can often be established to an ellipsoid accuracy better than ± 0.2 ft in X-Y-Z.

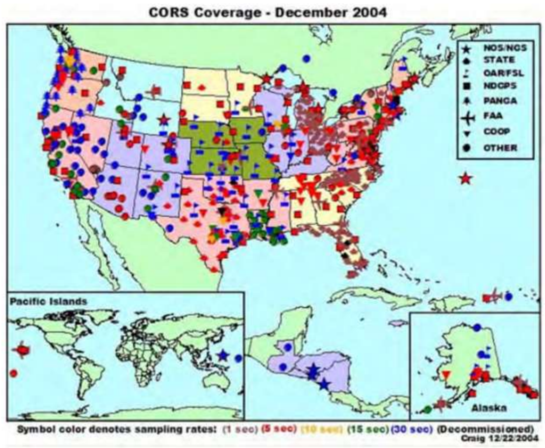


Figure 21: Continuously Operating Reference Stations as of 2001 (NGS).

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- e. International Terrestrial Reference Frame (ITRF).
  - The ITRF is a highly accurate geocentric reference frame with an origin at the center of the earth's mass.
  - The ITRF is continuously monitored and updated by the International Earth Rotation Service (IERS) using very-long-baseline-interferometry (VLBI) and other techniques.
  - These observations allow for the determination of small movements of fixed points on the earth's surface due to crustal motion, rotational variances, tectonic plate movement, etc.
  - These movements can average 10 to 20 mm/year in CONUS, and may become significant when geodetic control is established from remote reference stations.
  - These refinements can be used to accurately determine GPS positions observed on the basic WGS 84 reference frame.
  - NAD 83 coordinates are defined based on the ITRF year/epoch in which it is defined, e.g., ITRF 89, ITRF 96, ITRF 2000.
  - For highly accurate positioning where plate velocities may be significant, users should use the same coordinate reference frame and epoch for both the satellite orbits and the terrestrial reference frame.
  - USACE requirements for these precisions on control surveys would be rare, and would never be applicable to local facility mapping surveys.
  - Those obtaining coordinates from NGS datasheets must take care not to use ITRF values. T
  - he relationship between ITRF, NAD 83, and the geoid is illustrated in the figure on the following slide.

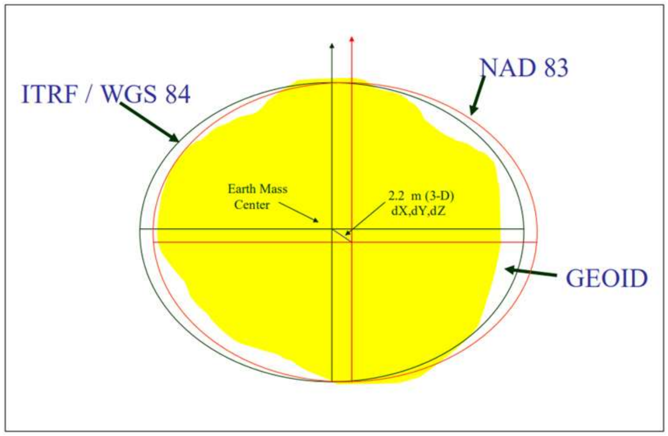
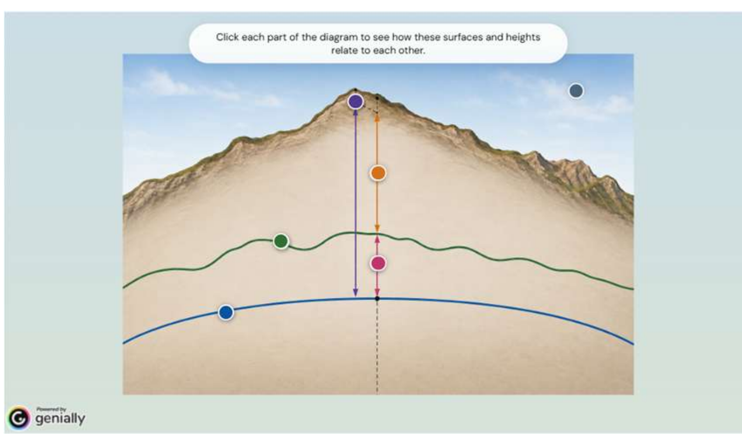


Figure 22: Relationship between ITRF, NAD 83, and the geoid.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

Check the next slide to explore an interactive GIF about the Anatomy of the Reference Ellipsoid



SECTION II. Horizontal Coordinate Systems

General

- Geocentric, geographic, or geodetic coordinates described above are rarely used to reference site plan topographic surveys or maps.
- Engineering site plan drawings are normally referenced to a local state plane coordinate system (SPCS), or in some cases a metric-based UTM system.
- In rare cases, they may be referenced to an arbitrary coordinate system relative to some point on the project--a monument, corner, road intersection, etc.
- In most cases, control surveys performed for setting project control will be computed and adjusted using the SPCS.
- The following paragraphs describe horizontal coordinate systems commonly used on facility site plan mapping and related control surveys.

Geographic Coordinates

- The use of geographic coordinates as a system of reference is accepted worldwide.
- It is based on the expression of position by latitude (parallels) and longitude (meridians) in terms of arc (degrees, minutes, and seconds) referred to the equator (north and south) and a prime meridian (east and west).
- The degree of accuracy of a geographic reference (GEOREF) is influenced by the map scale and the accuracy requirements for plotting and scaling.
- Examples of GEOREFs are as follows:

40° N 132° E (referenced to degrees of latitude and longitude).
40°21' N 132°14' E (referenced to minutes of latitude and longitude).
40°21'12" N 132°14'18" E (referenced to seconds of latitude and longitude).
40°21'12.4" N 132°14'17.7" E (referenced to tenths of seconds of latitude and longitude).
40°21'12.45" N 132°14'17.73" E (referenced to hundredths of seconds of latitude and longit

- US military maps and charts include a graticule (parallels and meridians) for plotting and scaling geographic coordinates.
- Graticule values are shown in the map margin.
- On maps and charts at scales of 1:250,000 and larger, the graticule may be indicated in the map interior by lines or ticks at prescribed intervals (for example, scale ticks and interval labeling at the corners of 1:50,000 at 1minute [in degrees, minutes, and seconds] and again every 5 minutes).

State Plane Coordinate Systems

- a. General.
  - State Plane Coordinate Systems (SPCS) were developed by the National Geodetic Survey (NGS) to provide plane coordinates over a limited region of the earth's surface.
  - To properly relate geodetic coordinates (φ-λ-h) of a point to a 2D plane coordinate representation (Northing, Easting), a conformal mapping projection must be used.
  - Conformal projections have mathematical properties that preserve differentially small shapes and angular relationships to minimize the errors in the transformation from the ellipsoid to the mapping plane.
  - Map projections that are most commonly used for large regions are based on either a conic or a cylindrical mapping surface.
  - The projection of choice is dependent on the north-south or east-west areal extent of the region.
  - Areas with limited east-west dimensions and indefinite north-south extent use the Transverse Mercator (TM) type projection.
  - Areas with limited north-south dimensions and indefinite east-west extent use the Lambert projection.
  - The SPCS is designed to minimize the spatial distortion at a given point to approximately one part in ten thousand (1:10,000).
  - To satisfy this criteria, the SPCS has been divided into zones that have a maximum width or height of approximately one hundred and fifty eight statute miles (158 miles).
  - Therefore, each state may have several zones or may employ both the Lambert (conic) and Transverse Mercator (cylindrical) projections.
  - The projection state plane coordinates are referenced to a specific geodetic datum (i.e. the datum that the initial geodetic coordinates are referenced to must be known).

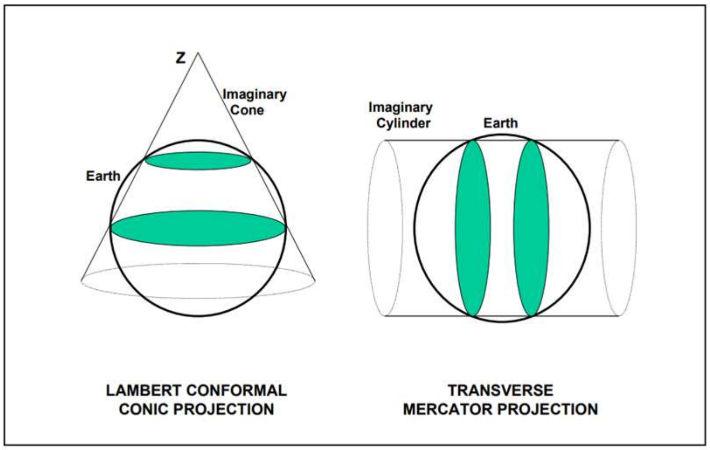


Figure 23: Common map projections.

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- b. Transverse Mercator (TM).
  - The Transverse Mercator projection uses a cylindrical surface to cover limited zones on either side of a central reference longitude.
  - Its primary axis is rotated perpendicular to the symmetry axis of the reference ellipsoid.
  - Thus, the TM projection surface intersects the ellipsoid along two lines equidistant from the designated central meridian longitude.
  - Distortions in the TM projection increase predominantly in the east-west direction.
  - The scale factor for the Transverse Mercator projection is unity where the cylinder intersects the ellipsoid.
  - The scale factor is less than one between the lines of intersection, and greater than one outside the lines of intersection.
  - The scale factor is the ratio of arc length on the projection to arc length on the ellipsoid.
  - To compute the state plane coordinates of a point, the latitude and longitude of the point and the projection parameters for a particular TM zone or state must be known.

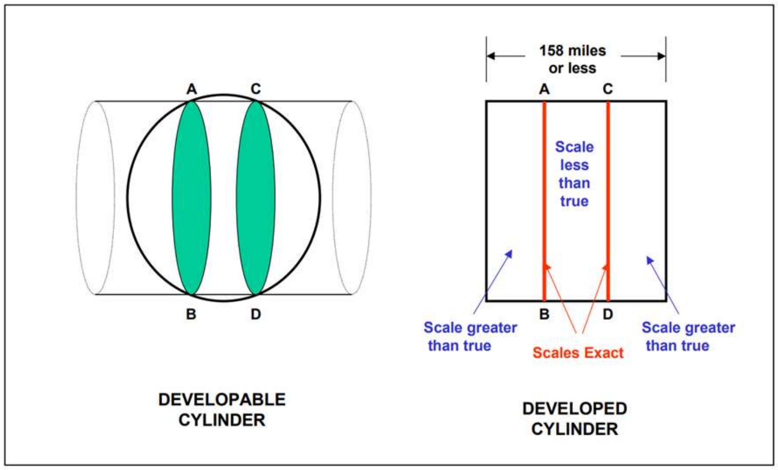


Figure 24: Transverse Mercator Projection.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

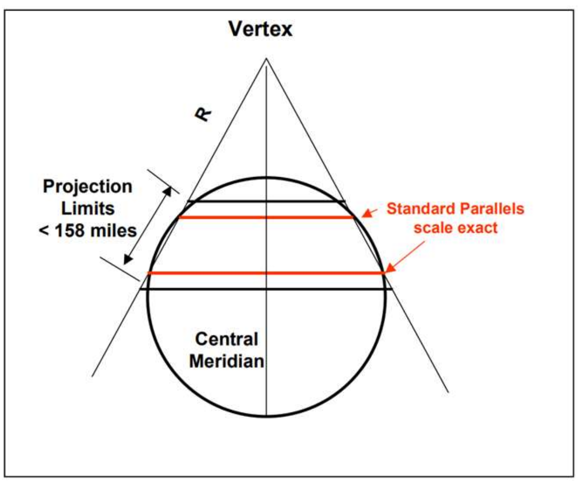


Figure 25: Lambert Projection.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- c. Lambert Conformal Conic (LCC).
  - The Lambert projection uses a conic surface to cover limited zones of latitude adjacent to two parallels of latitude.
  - Its primary axis is coincident with the symmetry axis of the reference ellipsoid.
  - Thus, the LCC projection intersects the ellipsoid along two standard parallels.
  - Distortions in the LCC projection increase predominantly in the north-south direction.
  - The scale factor for the Lambert projection is equal to unity at each standard parallel and is less than one inside, and greater than one outside the standard parallels.
  - The scale factor remains constant along the standard parallels.

- d. SPCS zones.
  - The figure below depicts the various SPCS zones in the US.
  - The unique state zone number provides a standard reference when using transformation software developed by NGS and the COE.
  - The state zone number remains constant in both NAD27 and NAD83 coordinate systems.

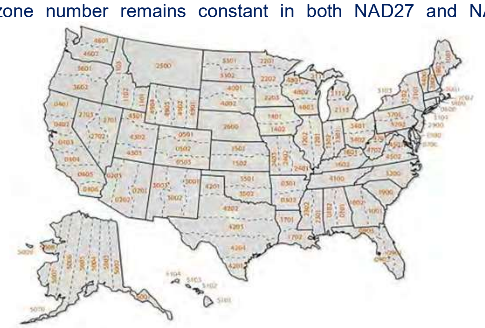


Figure 26: SPCS zones identification numbers for the various states.
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- e. Scale units.
  - State plane coordinates can be expressed in both feet and meters.
  - State plane coordinates defined on the NAD 27 datum are published in feet.
  - State plane coordinates defined on the NAD 83 datum are published in meters; however, state and federal agencies can request the NGS to provide coordinates in feet.
  - If NAD 83 based state plane coordinates are defined in meters and the user intends to convert those values to feet, the proper meter-feet conversion factor must be used.
  - Some states use the International Survey Foot rather than the US Survey Foot in the conversion of feet to meters.
    - International Survey Foot:
      - 1 International Foot = 0.3048 meter (exact)
    - US Survey Foot:
      - 1 US Survey Foot = 1200 / 3937 meter (exact)

Grid Elevations, Scale Factors, and Convergence

- In all planer grid systems, the grid projection only approximates the ellipsoid (or roughly the ground), and "ground-grid" corrections must be made for measured distances or angles (directions).
- Measured ground distances must be corrected for (1) elevation (sea level factor), and (2) ground to grid plane (scale factor).
- The figure on the following slide illustrates a reduction of a measured distance (D) down to the ellipsoid distance (S).
- Not shown is the subsequent reduction from the ellipsoid length to a grid system length.
- Observed directions (or angles) must also be corrected for grid convergence.
- Also shown on the figure is the relationship between ellipsoid heights (h), geoid heights (N), and orthometric heights (H).

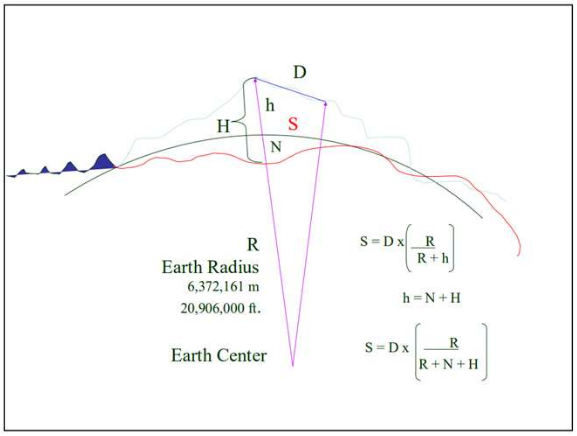


Figure 27: Reduction of measured slope distance D to ellipsoid distance S (NGS).

(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- a. Grid factor.
  - For most topographic surveys covering a small geographical site, these two factors can be combined into a constant "grid factor"--reference Kavanagh 1997:
    - Grid factor = Sea Level Factor x Scale Factor
  - then:
    - Ground Distance = Grid Distance / Grid Factor
  - or
    - Grid Distance = Ground Distance x Grid Factor

- b. Convergence.
  - Between two fixed points, the geodetic azimuth will differ from the grid azimuth.
  - This difference is known as "convergence" and varies with the distance from the central meridian of the projection.
  - Thus, if a geodetic azimuth is given between two fixed points (inversed from published geographic coordinates, astronomic, or GPS), then it must be corrected for convergence to obtain an equivalent grid azimuth.
  - If lengthy control traverses are being computed on a SPCS or UTM grid, then additional second term corrections to observed angles may be required.

- c. Use of data collectors.
  - The above grid corrections should rarely have to be performed when modern survey data collectors are being used.
  - These total station or RTK data collectors (with full COGO and adjustment capabilities) will automatically perform all the necessary geographic to grid coordinate translations, including sea level reductions and local grid system conversions that are later transformed and adjusted into an established SPCS grid at a true elevation.
  - If for some reason you are not using a data collector that seamlessly performs these translation functions, and you are performing a survey in higher elevations, then you must correct original distance observations for the sea level reduction.
  - If you transfer these observed distances and angles to a SPCS or UTM grid, then you must correct for grid scale factor and convergence described above.
  - Consult any of the referenced surveying textbooks at Appendix A-2 for procedures and examples.

Universal Transverse Mercator Coordinate System

- Universal Transverse Mercator (UTM) coordinates are used in surveying and mapping when the size of the project extends through several state plane zones or projections.
- UTM coordinates are also utilized by the US Army, Air Force, and Navy for tactical mapping, charting, and geodetic applications.
- It may also be used to reference site plan engineering surveys if so requested in CONUS or OCONUS installations.
- The UTM projection differs from the TM projection in the scale at the central meridian, origin, and unit representation.
- The scale at the central meridian of the UTM projection is 0.9996.
- In the Northern Hemisphere, the northing coordinate has an origin of zero at the equator.
- In the Southern Hemisphere, the southing coordinate has an origin of ten million meters (10,000,000 m).
- The easting coordinate has an origin five hundred thousand meters (500,000 m) at the central meridian.
- 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. UTM coordinates are always expressed in meters.
- USACE program CORPSCON can be used to transform coordinates between UTM and SPCS systems.

The US Military Grid-Reference System

- The US Military Grid-Reference System (MGRS) is designed for use with UTM grids.
- For convenience, the earth is generally divided into 6° by 8° geographic areas, each of which is given a unique grid-zone designation.
- These areas are covered by a pattern of 100,000-meter squares.
- Two letters (called the 100,000-meter-square letter identification) identify each square.
- This identification is unique within the area covered by the grid-zone designation.

- a. The MGRS is an alphanumeric version of a numerical UTM grid coordinate.
  - Thus, for that portion of the world where the UTM grid is specified (80° south to 84° north), the UTM grid-zone number is the first element of a military grid reference.
  - This number sets the zone longitude limits.
  - The next element is a letter that designates a latitude bond.
  - Beginning at 80° south and proceeding northward, 20 bands are lettered C through X.
  - In the UTM portion of the MGRS, the first three characters designate one of the areas within the zone dimensions.

- b. A reference that is keyed to a gridded map (of any scale) is made by giving the 100,000-metersquare letter identification together with the numerical location.
  - Numerical references within the 100,000-meter square are given to the desired accuracy in terms of the easting and northing grid coordinates for the point.

- c. The final MGRS position coordinate consists of a group of letters and numbers that include the following elements:
  - The grid-zone designation.
  - The 100,000-meter-square letter identification.
  - The grid coordinates (also referred to as rectangular coordinates) of the numerical portion of the reference, expressed to a desired refinement.
  - The reference is written as an entity without spaces, parentheses, dashes, or decimal points.

- Examples are as follows:
  - 18S (locating a point within the grid-zone designation).
  - 18SUU (locating a point within a 100,000-meter square).
  - 18SUU80 (locating a point within a 10,000-meter square).
  - 18SUU8401 (locating a point within a 1,000-meter square).
  - 18SUU836014 (locating a point within a 100-meter square).

- d. To satisfy special needs, a reference can be given to a 10-meter square and a 1-meter square.
  - Examples are as follows:
    - 8SUU83630143 (locating a point within a 10-meter square).
    - 18SUU8362601432 (locating a point within a 1-meter square).
- e. There is no zone number in the polar regions.
  - A single letter designates the semicircular area and the hemisphere.
  - The letters A, B, Y, and Z are used only in the polar regions.
  - An effort is being made to reduce the complexity of grid reference systems by standardizing a single, worldwide grid reference system.

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GEODETIC REFERENCE DATUMS AND LOCAL COORDINATE SYSTEMS (CONT'D)

Control and Topographic Surveying - Part 2
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US National Grid System

- A US National Grid (USNG) system has been developed to improve public safety, commerce, and aid the casual GPS user with an easy to use geoaddress system for identifying and determining location with the help of a USNG gridded map and/or a USNG enabled GPS system.
- The USNG can provide for whatever level of precision is desired.
- Many users may prefer to continue using the UTM format for applications requiring precision greater than 1 meter.

- a. Grid Zone Designation (GZD).
  - The US geographic area is divided into 6-degree longitudinal zones designated by a number and 8-degree latitudinal bands designated by a letter.
  - Each area is given a unique alphanumeric Grid Zone Designator--e.g., 18S.
- b. 100,000-meter square identification.
  - Each GZD 6x8 degree area is covered by a specific scheme of 100,000-meter squares where each square is identified by two unique letters--e.g., 18SUJ identifies a specific 100,000-meter square in the specified GZD.

- c. Grid coordinates.
  - A point position within the 100,000-meter square shall be given by the UTM grid coordinates in terms of its Easting (E) and Northing (N).
  - An equal number of digits shall be used for E and N where the number of digits depends on the precision desired in position referencing.
  - In this convention, the reading shall be from left with Easting first and then Northing.
  - Examples:
    - 18SUJ20 - Locates a point with a precision of 10 km
    - 18SUJ2306 - Locates a point with a precision of 1 km
    - 18SUJ234064 - Locates a point with a precision of 100 meters
    - 18SUJ23480647 - Locates a point with a precision of 10 meters
    - 18SUJ2348306479 - Locates a point with a precision of 1 meter
  - The number of digits in Easting and Northing can vary, depending on specific requirements or application.

Chainage-Offset Coordinate Systems

- Most linear engineering and construction projects (roads, railways, canals, navigation channels, levees, floodwalls, beach renourishment, etc.) are locally referenced using the traditional engineering chainage-offset system.
- Usually, SPCS coordinates are provided at the PIs, from which (given the alignment between PIs) a SPCS coordinate can then be computed for any given station-offset point.
- Chainage-offset systems are used for locating cross-sections along even centerline stations.
- Topographic elevation and feature data is then collected along each section relative to the centerline.
- Likewise, road, canal, levee alignments can be staked out relative to station-offset parameters, and internally in a total station or RTK system data collector, these offsets may actually be transformed from a SPCS.

- a. Station.
  - Alignment stationing (or chainage) zero references are arbitrarily established for a given project or sectional area.
  - Stationing on a navigation project usually commences offshore on coastal projects and runs inland or upstream.
  - Stationing follows the channel centerline alignment.
  - Stationing may be accumulated through each PI or zero out at each PI or new channel reach.
  - Separate stationing is established for widener sections, turning basins, levees, floodwalls, etc.
  - Each district may have its own convention.
  - Stationing coordinates use "+" signs to separate the second- and third-place units (XXX + XX.XX).
  - Metric chainage often separates the third and fourth places (XXX + XXX.XX) to distinguish the units from English feet; however, some districts use this convention for English stationing units.

- b. Offsets.
  - Offset coordinates are distances from the centerline alignment.
  - Offsets carry plus/minus coordinate values.
  - Normally, offsets are positive to the right (looking toward increasing stationing).
  - Some USACE Districts designate cardinal compass points (east-west or north-south) in lieu of a coordinate sign.
  - On some navigation projects, the offset coordinate is termed a "range," and is defined relative to the project centerline or, in some instances, the channel-slope intersection line (toe).
  - Channel or canal offsets may be defined relative to a fixed baseline on the bank or levee.

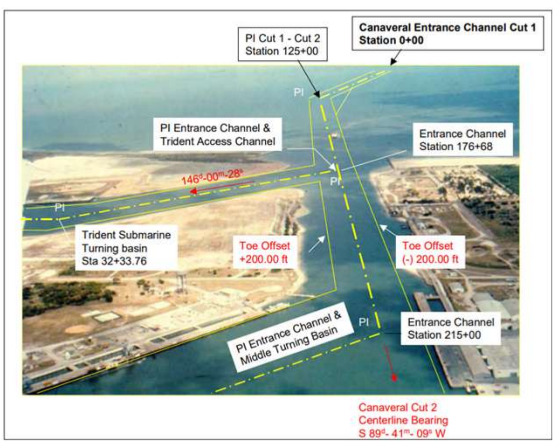


Figure 28: Chainage-offset project control scheme for a typical deep-draft navigation project--Cape Canaveral, FL (Jacksonville District).
(Source: US Army Corps of Engineers, "Control and Topographic Surveying," 1 January 2007, EM 1110-1-1005)

- c. Azimuth.
  - Azimuths are computed relative to the two defining PIs.
  - Either 360-deg azimuth or bearing designations may be used.
  - Azimuths should be shown to the nearest second.

- d. Other local alignments.
  - Different station-offset reference grids may be established for individual portions of a project.
  - River sections and coastal beach sections are often aligned perpendicular to the project/coast.
  - Each of these sections is basically a separate local datum with a different reference point and azimuth alignment.
  - Beach sections may also be referenced to an established coastal construction setback line.
  - Circular and transition (spiral) curve alignments are also found in some rivers, canals, and flood control projects such as spillways and levees.
  - Surveys will generally be aligned to the chainage and offsets along such curves.
  - Along inland waterways, such as the Mississippi River, stationing is often referenced to either arbitrary or monumented baselines along the bank.

- In many instances, a reference baseline for a levee is used, and surveys for revetment design and construction are performed from offsets to this line.
- Separate baselines may exist over the same section of river, often from levees on opposite banks or as the result of revised river flow alignments.
- Baseline stationing may increase either upstream or downstream.
- Most often, the mouth of a river is considered the starting point (Station 0 + 00), or the river reaches are summed to assign a station number at the channel confluence.
- Stationing may increase consecutively through PIs or reinitialize at channel turns.
- In addition, supplemental horizontal reference may also be made to a river mile designation system.
- River mile systems established years ago may no longer be exact if the river course has subsequently realigned itself.
- River mile designations can be used to specify geographical features and provide navigation reference for users.

Datum Conversions and Transformation Methods

- a. General.
  - Federal Geodetic Control Subcommittee (FGCS) members, which includes USACE, have adopted NAD 83 as the standard horizontal datum for surveying and mapping activities performed or financed by the Federal government.
  - To the extent practicable, legally allowable, and feasible, USACE should use NAD 83 in its surveying and mapping activities.
  - Transformations between NAD 27 coordinates and NAD 83 coordinates are generally obtained using the CORPS Convert (CORPSCON) software package or other North American Datum Conversion (e.g., NADCON) based programs.

- b. Conversion techniques.
  - USACE survey control published in the NGS control point database has been already converted to NAD 83 values.
  - However, most USACE survey control was not originally in the NGS database and was not included in the NGS readjustment and redefinition of the national geodetic network.
  - Therefore, USACE will have to convert this control to NAD 83.
  - Coordinate conversion methods considered applicable to USACE projects are discussed below.

- (1) Resurvey from NAD 83 Control.
  - A new survey using NGS published NAD 83 control could be performed over the entire project.
  - This could be either a newly authorized project or one undergoing major renovation or maintenance.
  - Resurvey of an existing project must tie into all monumented points.
  - Although this is not a datum transformation technique, and would not normally be economically justified unless major renovation work is being performed, it can be used if existing NAD 27 control is of low density or accuracy.

- (2) Readjustment of Survey.
  - If the original project control survey was connected to NGS control stations, the survey may be readjusted using the NAD 83 coordinates instead of the NAD 27 coordinates originally used.
  - This method involves locating the original field notes and observations, and completely readjusting the survey and fixing the published NAD 83 control coordinates.

- (3) Mathematical Transformations.
  - Since neither of the above methods can be economically justified on most USACE projects, mathematical approximation techniques for transforming project control data to NAD 83 have been developed.
  - These methods yield results which are normally within ± 1 ft of the actual values and the distribution of errors are usually consistent within a local project area.
  - Since these coordinate transformation techniques involve approximations, they should be used with caution when real property demarcation points and precise surveying projects are involved.
  - When mathematical transformations are employed they should be adequately noted so that users will be aware of the conversion method.

- c. Horizontal datum transformation methods.
  - Coordinate transformations from one geodetic reference system to another can be most practically made either by using a local seven-parameter transformation or by interpolation of datum shift values across a given region.
  - (1) Seven parameter transformations.
  - (2) Grid-shift transformations.
    - Current methods for interpolation of datum shift values use the difference between known coordinates of common points from both the NAD 27 and NAD 83 adjustments to model a best-fit shift in the regions surrounding common points.
    - A grid of approximate datum shift values is established based on the computed shift values at common points in the geodetic network.
    - The datum shift values of an unknown point within a given grid square are interpolated along each axis to compute an approximate shift value between NAD 27 and NAD 83.
    - Any point that has been converted by such a transformation method should be considered as having only approximate NAD 83 coordinates.

- (3) NADCON/CORPSCON.
  - NGS developed the transformation program NADCON, which yields consistent NAD 27 to NAD 83 coordinate transformation results over a regional area.
  - This technique is based on the above grid-shift interpolation approximation.
  - NADCON was reconfigured into a more comprehensive program called CORPSCON.
  - Technical documentation and operating instructions for CORPSCON can be obtained at the following ERDC Web site.
  - This software converts between:

NAD 27	NAD 83	SPCS 27	SPCS 83
UTM 27	UTM 83	NGVD 29	NAVD 88
GEOID03	HARN		

- Since the overall CORPSCON datum shift (from point to point) varies throughout North America, the amount of datum shift across a local project is also not constant.
- The variation can be as much as 0.1 ft per mile.
- Examples of some NAD 27 to NAD 83 based coordinate shift variations that can be expected over a 10,000-ft section of a project are shown below:

Project Area	SPCS Reference	Per 10,000 feet
Baltimore, MD	1900	0.16 ft
Los Angeles, CA	0405	0.15 ft
Mississippi Gulf Coast	2301	0.08 ft
Mississippi River (IL)	1202	0.12 ft
New Orleans, LA	1702	0.22 ft
Norfolk, VA	4502	0.08 ft
San Francisco, CA	0402	0.12 ft
Savannah, GA	1001	0.12 ft
Seattle, WA	4601	0.10 ft

- Such local scale changes will cause project alignment data to distort by unequal amounts.
- Thus, a 10,000- ft tangent on NAD 27 project coordinates could end up as 9,999.91 feet after mathematical transformation to NAD 83 coordinates.
- Although such differences may not be appear significant from a lower-order construction survey standpoint, the potential for such errors must be recognized.
- Therefore, the transformations will not only significantly change absolute coordinates on a project and the datum transformation process will slightly modify the project's design dimensions and/or construction orientation and scale.
- For example, on a navigation project, an 800.00-ft wide channel could vary from 799.98 to 800.04 feet along its reach.
- This variation could also affect grid alignment azimuths.

- Moreover, if the local SPCS 83 grid was further modified, then even larger dimension changes can result.
- Correcting for distortions may require recomputation of coordinates after conversion to ensure original project dimensions and alignment data remain intact.
- This is particularly important for property and boundary surveys.
- A less accurate alternative is to compute a fixed shift to be applied to all data points over a limited area.
- Determining the maximum area over which such a fixed shift can be applied is important.
- Computing a fixed conversion factor with CORPSCON can be made to within ±1 foot.
- Typically, this fixed conversion would be computed at the center of a sheet or at the center of a project and the conversions in X and Y from NAD 27 to NAD 83 and from SPCS 27 to SPCS 83 indicated by notes on the sheets or data sets.
- Since the conversion is not constant over a given area, the fixed conversion amounts must be explained in the note.

- The magnitude of the conversion factor change across a sheet is a function of location and the drawing scale.
- Whether the magnitude of the distortion is significant depends on the nature of the project.
- For example, a 0.5-ft variation on an offshore navigation project may be acceptable for converting depth sounding locations, whereas a 0.1-ft change may be intolerable for construction layout on an installation.
- In any event, the magnitude of this gradient should be computed by CORPSCON at each end (or corners) of a sheet or project.
- If the conversion factor variation exceeds the allowable tolerances, then a fixed conversion factor should not be used.

- Two examples of using Fixed Conversion Factors follow:
  - Example 1.
    - ✓ Assume a 1 inch = 40 ft scale site plan map on existing SPCS 27 (VA South Zone 4502).
    - ✓ Using CORPSCON, convert existing SPCS 27 coordinates at the sheet center and corners to SPCS 83 (US Survey Foot), and compare SPCS 83-27 differences.

SPCS 83		SPCS 27	SPCS 83 - SPCS 27
Center of Sheet	N 3,527,095.554 E 11,921,022.711	Y 246,200.000 X 2,438,025.000	dY = 3,280,895.554 dX = 9,482,997.711
NW Corner	N 3,527,595.553 E 11,920,522.693	Y 246,700.000 X 2,437,525.000	dY = 3,280,895.553 dX = 9,482,997.693
NE Corner	N 3,527,595.556 E 11,921,522.691	Y 246,700.000 X 2,438,525.000	dY = 3,280,895.556 dX = 9,482,997.691
SE Corner	N 3,526,595.535 E 11,921,522.702	Y 245,700.000 X 2,438,525.000	dY = 3,280,895.535 dX = 9,482,997.702
SW Corner	N 3,526,595.535 E 11,920,522.704	Y 245,700.000 X 2,437,525.000	dY = 3,280,895.535 dX = 9,482,997.704

- ✓ Since coordinate differences do not exceed 0.03 feet in either the X or Y direction, the computed SPCS 83-27 coordinate differences at the center of the sheet may be used as a fixed conversion factor to be applied to all existing SPCS 27 coordinates on this drawing.

- ❖ Example 2.
  - ✓ Assuming a 1inch = 1,000 ft base map is prepared of the same general area, a standard drawing will cover some 30,000 feet in an east-west direction.
  - ✓ Computing SPCS 83-27 differences along this alignment yields the following:

SPCS 83		SPCS 27	SPCS 83 - SPCS 27
West End	N 3,527,095.554 E 11,921,022.711	Y 246,200.000 X 2,438,025.000	dY = 3,280,895.554 dX = 9,482,997.711
East End	N 3,527,095.364 E 11,951,022.104	Y 246,200.000 X 2,468,025.000	dY = 3,280,895.364 dX = 9,482,997.104

- ✓ The conversion factor gradient across this sheet is about 0.2 ft in Y and 0.6 ft in X.
- ✓ Such small changes are not significant at the plot scale of 1 inch = 1,000 ft; however, for referencing basic design or construction control, applying a fixed shift across an area of this size is not recommended -- individual points should be transformed separately.
- ✓ If this 30,000-ft distance were a navigation project, then a fixed conversion factor computed at the center of the sheet would suffice for all bathymetric features.
- ✓ Caution should be exercised when converting portions of projects or military installations or projects that are adjacent to other projects that may not be converted.
- ✓ If the same monumented control points are used for several projects or parts of the same project, different datums for the two projects or parts thereof could lead to surveying and mapping errors, misalignment at the junctions and layout problems during construction.

- d. Dual grids ticks.
  - Depicting both NAD 27 and NAD 83 grid ticks and coordinate systems on maps and drawings should be avoided where possible.
  - This is often confusing and can increase the chance for errors during design and construction.
  - However, where use of dual grid ticks and coordinate systems is unavoidable, only secondary grid ticks in the margins will be permitted.

- e. Field survey methods.
  - If GPS is used to set new control points referenced to higher order control many miles from the project (e.g., CORS networks), inconsistent data may result at the project site.
  - If the new control is near older control points that have been converted to NAD 83 using CORPSCON, two slightly different network solutions can result, even though both have NAD 83 coordinates.
  - In order to avoid these situations, it is recommended that all project control (old and new) be tied into the same reference system--preferably the NSRS.

- f. Local project datums.
  - Local project datums that are not referenced to NAD 27 cannot be mathematically converted to NAD 83 with CORPSCON.
  - Field surveys connecting them to other stations that are referenced to NAD 83 are required.

Horizontal Transition Plan from NAD 27 to NAD 83

- a. General.
  - Not all maps, engineering site drawings, documents, and associated products containing coordinate information will require conversion to NAD 83.
  - To insure an orderly and timely transition to NAD 83 is achieved for the appropriate products, the following general guidelines should be followed:
    - (1) Initial surveys.
      - All initial surveys should be referenced to NAD 83.
    - (2) Active projects.
      - Active projects where maps, site drawings or coordinate information are provided to non-USACE users (e.g., NOAA, USCG, FEMA, and others in the public and private sector) coordinates should be converted to NAD 83 the next time the project is surveyed or maps or site drawings are updated for other reasons.

- (3) Inactive projects.
  - For inactive projects or active projects where maps, site drawings or coordinate information are not normally provided to non-USACE users, conversion to NAD 83 is optional.
- (4) Datum notes.
  - Whenever maps, site drawings or coordinate information (regardless of type) are provided to non-USACE users, it should contain a datum note, such as the following:

THE COORDINATES SHOWN ARE REFERENCED TO NAD *[27/83] AND ARE IN FEET BASED ON THE SPCS *[27/83] *[STATE, ZONE]. DIFFERENCES BETWEEN NAD 27 AND NAD 83 AT THE CENTER OF THE *[SHEET/DATASET] ARE *[dLat, dLon, dX, dY]. DATUM CONVERSION WAS PERFORMED USING THE COMPUTER PROGRAM "CORPSCON." METRIC CONVERSIONS WERE BASED ON THE *[US SURVEY FOOT = 1200/3937 METER] [INTERNATIONAL FOOT = 30.48/100 METER].

- b. Levels of effort.
  - For maps and site drawings the conversion process entails one of three levels of effort:
    - (1) Conversion of coordinates of all mapped details to NAD 83, and redrawing the map,
    - (2) Replace the existing map grid with a NAD 83 grid,
    - (3) Simply adding a datum note.
  - For surveyed points, control stations, alignment, and other coordinated information, conversion must be made either through a mathematical transformation or through readjustment of survey observations.

- c. Detailed instructions.
  - (1) Initial surveys on Civil Works projects.
    - The project control should be established on NAD 83 relative to NGS's National Spatial Reference System (NSRS) using conventional or GPS surveying procedures.
    - The local SPCS 83 grid should be used on all maps and site drawings.
    - All planning and design activities should then be based on the SPCS 83 grid.
    - This includes supplemental site plan mapping, core borings, project design and alignment, construction layout and payment surveys, and applicable boundary or property surveys.
    - All maps and site drawings shall contain datum notes.
    - If the local sponsor requires the use of NAD 27 for continuity with other projects that have not yet converted to NAD 83, conversion to NAD 27 could be performed using the CORPSCON transformation techniques.

- (2) Active Civil Works Operations and Maintenance projects undergoing maintenance or repair.
  - These projects should be converted to NAD 83 during the next maintenance or repair cycle in the same manner as for newly initiated civil works projects.
  - However, if resources are not available for this level of effort, either redraw the grids or add the necessary datum notes.
  - Plans should be made for the full conversion during a later maintenance or repair cycle when resources can be made available.

- (3) Military Construction and master planning projects.
  - All installations and master planning projects should remain on NAD 27 or the current local datum until a thoroughly coordinated effort can be arranged with the MACOM and installation.
  - An entire installation's control network should be transformed simultaneously to avoid different datums on the same installation.
  - The respective MACOMs are responsible for this decision.
  - However, military operations may require NAD 83, including SPCS 83 or UTM metric grid systems.
  - If so, these shall be performed separate from facility engineering support.
  - A dual grid system may be required for such operational applications when there is overlap with normal facilities engineering functions.
  - Coordinate transformations throughout an installation can be computed using the procedures described herein.
  - Care must be taken when using transformations from NAD 27 with new control set using GPS methods from points remote from the installation.

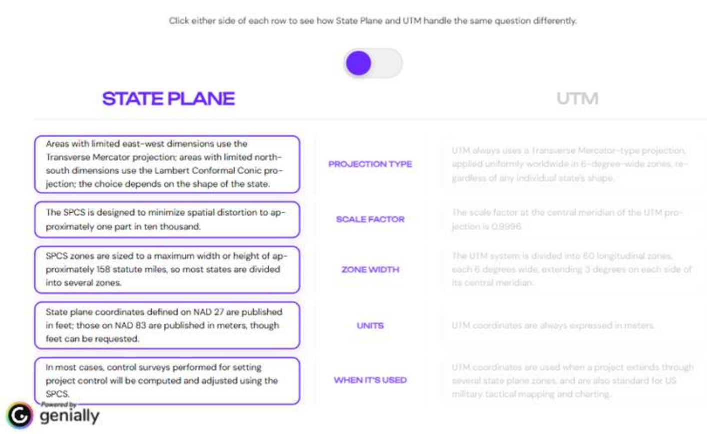
- (4) Real Estate.
  - Surveys, maps, and plats prepared in support of civil works and military real estate activities should conform as much as possible to state requirements.
  - Since most states have adopted NAD 83, most new boundary and property surveys should be based on NAD 83.
  - The local authorities should be contacted before conducting boundary and property surveys to ascertain their policies.
  - It should be noted that several states have adopted the International Foot for their standard conversion from meters to feet.
  - In order to avoid dual coordinates on USACE survey control points that have multiple uses, all control should be based on the US Survey Foot, including control for boundary and property surveys.
  - In states where the International Foot is the only accepted standard for boundary and property surveys, conversion of these points to NAD 83 should be based on the International foot, while the control remains based on the US Survey foot.

- (5) Regulatory functions.
  - Surveys, maps, and site drawings prepared in support of regulatory functions should begin to be referenced to NAD 83 unless there is some compelling reason to remain on NAD 27 or locally used datum.
  - Conversion of existing surveys, maps, and drawings to NAD 83 is not necessary.
  - Existing surveys, maps, and drawings need only have the datum note added before distribution to non-USACE users.
  - The requirements of local, state and other Federal permitting agencies should be ascertained before site specific conversions are undertaken.
  - If states require conversions based on the International foot, the same procedures as described above for Real Estate surveys should be followed.

- (6) Other existing projects.
  - Other existing projects, e.g., beach nourishment, submerged offshore disposal areas, historical preservation projects, etc., need not be converted to NAD 83.
  - However, existing surveys, maps, and drawings should have the datum note added before distribution to non-USACE users.

- (7) Work for others.
  - Existing projects for other agencies will remain on NAD 27 or the current local datum until a thoroughly coordinated effort can be arranged with the sponsoring agency.
  - The decision to convert rests with the sponsoring agency.
  - However, existing surveys, maps, and drawings should have the datum note added before distribution to non-USACE users.
  - If sponsoring agencies do not indicate a preference for new projects, NAD 83 should be used.
  - The same procedures as described previously for initial surveys on Civil Works projects should be followed.

Check the next slide to explore an interactive GIF about State Plane versus UTM



THANK YOU FOR LISTENING!

This concludes our presentation, and we hope that you enjoyed it.

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REFERENCES

This interactive presentation was adapted from Section II of Chapter 3, Chapter 4, and Sections I and II of Chapter 5 of the document by the US Army Corps of Engineers, Control and Topographic Surveying, January 2007, EM 1110-1-1005.

COURSE PROVIDER BIOGRAPHY

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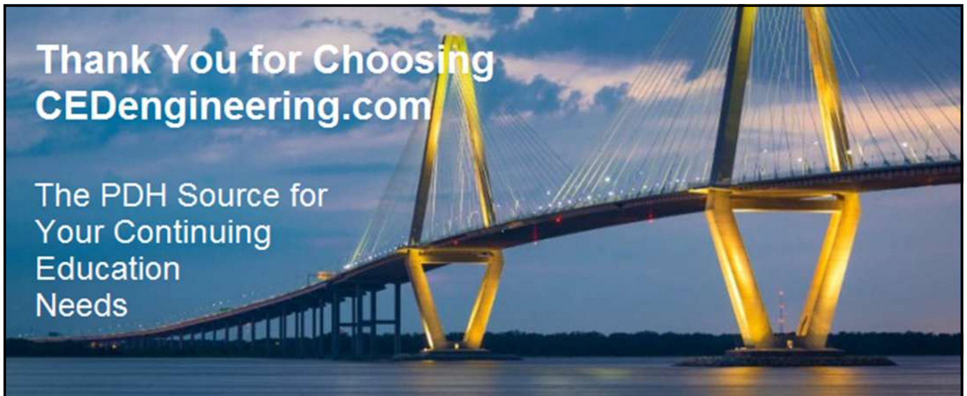

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