Chapter 2
Control Surveys and State Plane Coordinate Systems

Chapter 2 Control Surveys and State Plane Coordinate Systems

Although most engineering related surveys seem to be made on a plane, they are actually performed on the curved surface of the Earth. To ensure continuity and congruity of statewide projects, it is necessary to register the survey points to a mathematical representation of the topography (actual surface of the earth).

The objective of this chapter is to introduce the key terminology and concepts necessary to represent the actual surface of the earth. The first concept is the mathematical representation of the earth as a curved surface and its approximation as a plane. This concept leads to understanding the New Jersey State Plane Coordinate System (NJ-SPCS) and its relationship to the topography where surveys are performed. A second concept that will be introduced is elevation. Following the introduction of these concepts, methods for performing control surveys will be presented. Finally, the importance of monumentation of survey points will be discussed.

2.1 Survey Datums and Coordinate Systems

2.1.1 Introduction

A datum is any numerical or geographical quantity or set of such quantities, which may serve as a reference or base for other quantities. In surveying (or geodesy), datum is a set of quantities that describe the shape, size, rotation and gravity field parameters of the Earth. A geodetic datum is comprised of two surfaces that approximate the real Earth. The first of these surfaces is an ellipsoid (or spheroid) that mathematically approximates the shape and size of the Earth. The ellipsoid is used as a reference frame for horizontal position computations. The second surface is the geoid that is a mathematically irregular surface of elevation zero. The geoid is used for determining elevations.

In order to design and construct highway projects, all distances (both horizontal and vertical) must be referenced to a recognized datum. It is recommended that all projects be referenced to the horizontal North American Datum of 1983 (NAD 83) and that the vertical component of the project be referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29) or the North American Vertical Datum of 1988 (NAVD 88). The datum used should be clearly noted on the plans.

2.1.2 Horizontal Datums

A horizontal datum is a surface of constant values which forms the basis for the computations of horizontal control surveys in which the curvature of the earth is considered. A reference ellipsoid is used for a horizontal datum. Five parameters are required to define a horizontal datum: two to specify the dimensions of the ellipsoid, two to specify the location of an initial point (origin), and one to specify the orientation (i.e. north) of the coordinate system. Selection of datum parameters depends on the objectives of the datum. Countries, or groups of countries, choose different reference ellipsoids to minimize the deviations between the topography and the ellipsoid in their region (best-fit ellipsoid for their region.). These individual geodetic datums are often referred to as local datums. Since the advent of GPS, the tendency is to adopt a global datum that provides the best fit for the entire Earth. Global datums are also related to the Earth's centermass, the geocenter.

Positions of points (coordinates) on a horizontal datum are expressed in terms of latitude and longitude. Latitude is measured, in degrees of arc, along a north-south direction from the plane of the equator to the location of a point on the ellipsoid. At the equator the latitude is 0° while at the north and south poles the latitude is 90°N and 90°S respectively. Longitude is measured, in degrees of arc, in an east-west direction from a reference meridian (customarily Greenwich meridian) to the local meridian. A meridian at any given point (called a local meridian) is an arc on the ellipsoid that connects the north and south poles and passes through that point. In the US, longitudes are measured in degrees of arc from Greenwich in a westerly direction. For example, the longitude at center of New Jersey is approximately 74.5°W.

Modern positioning equipment, such as GPS, display locations in terms of latitudes and longitudes. USGS topographic maps, as well as many navigation charts, include geodetic latitude and longitude in the form of grid tics along their margins.

A discussion of the calculations for the determination of position by the use of geodetic or spherical coordinates is rather complex and is not essential to the purpose of this manual.

2.1.2.1 The North American Datum of 1927 (NAD27)

In 1927 a general adjustment of all the horizontal geodetic surveys was performed. The adjustment utilized the Clarke ellipsoid of 1866 and held fixed the latitude and longitude of station Meades Ranch in Kansas as the initial point, along with an azimuth to a nearby station Waldo. The control points that made up the national geodetic reference system were assigned NAD27 latitudes and longitudes. Many local surveys were based on these control points and their positions were also expressed in terms of NAD27 coordinates. The introduction of highly accurate electronic measurement systems in the late 1950s, and the advent of satellite tracking systems such as Doppler (an earlier satellite positioning system) in the 1960s and GPS in 1970s, unveiled many weaknesses in NAD 27. Discrepancies between existing control and newly established surveys necessitated the establishment of an entirely new datum, rather than fixing NAD 27.

2.1.2.2 The North American Datum Of 1983 (NAD83)

In 1986 the National Geodetic Survey (NGS) completed a project for the redefinition and adjustment of the existing horizontal reference system. The North American Datum of 1983 (NAD83) represents the single most accurate, and comprehensive geodetic survey datum in the history of the United States. It supersedes the North American Datum of 1927 (NAD 27). The new datum, NAD83, is earth centered, and relies on an ellipsoid (and other constants) of the Geodetic Reference System of 1980 (GRS 80). The primary advantage of GRS 80 is that it facilitates the computation of correct geometric relationships on a global, as well as a continental, scale.

The adjusted latitudes and longitudes of all monuments in NAD83 differ from their NAD27 values. The differences are due to change in datum, as well as due to superior (quantitative and qualitative) measurements and adjustment.

One should be very careful not to mix latitudes and longitudes between these different datums.

2.1.2.3 HARN and NAD83

The original NAD 83 geodetic network was computed mostly by using traditional surveying observations and methods. Very few GPS observations were included in the adjustment computation. The design and implementation of this network preceded the developments of the GPS technology and, therefore, the practical usage of these control points for GPS application can be problematic. Some of these problems are:

1. Most of these control points are not "GPSable”. In other words, the points are located near objects that obstruct the required clear visibility between the receiver and the satellites.

2. Many of these control points are located on mountain tops and other locations that are not easily accessible. To work efficiently with GPS, one needs to have quick and easy access to control points.

3. Control points of the original NAD 83 network are spaced irregularly. Hence, chances are that there will be insufficient control points in the vicinity of your project.

4. The original NAD 83 network is not accurate enough to serve as control for GPS observations. The most accurate horizontal standard in the original NAD 83 network is 1:100,000 as compared to a 1:100,000,000 accuracy attainable by GPS.

To eliminate or significantly reduce the problems listed above several states (including New Jersey) in conjunction with National Geodetic Survey are developing a High Accuracy Reference Network (HARN). The HARN is designed to establish geodetic control points accessible 24 hours a day by car or light truck within, at most, 30 to 45 minutes from anywhere in the State. Once the HARN is established, a new adjustment must be computed and the points in the network are assigned new coordinates. These coordinates are different from those of the original NAD 83 adjustment. The new coordinate values will be designated as NAD 83(199x), where 'x' indicates the year of the adjustment. For example, a point that was computed in 1994 bears an NAD 83(1994) code value.

NAD 83(199x) is not a new datum. It is just an improvement over the original NAD83 datum. When the HARN project is completed for the entire US, NGS will most probably embark on defining and computing a new datum.

2.1.3 Vertical Datums

Elevations for engineering projects must be referenced to a single vertical datum so various phases of a project, and contiguous projects, will match. This datum can be based on some particular standard, such as sea level, an assumed elevation or the elevation of a local permanent point or natural object. Various organizations, private and public, use datums that best serve their individual needs. This has led to many different datums throughout the State, causing a considerable amount of confusion.

The New Jersey Department of Transportation has adopted the use of the NGVD 29 or NAVD 88 datum as established by the NGS as the vertical datum for highway projects. The datum used must be noted on each benchmark note/description, as the datums are not the same. Exceptions to NGVD 29 or NAVD 88 may be permitted for small, remote, isolated surveys where ties to a recognized vertical datum cannot be economically established.

2.1.3.1 The National Geodetic Vertical Datum (NGVD 29)

The orthometric heights in NGVD 29 refer to the geoid, and are usually referred to as MSL heights (mean sea level heights). Mean sea level is the average height of the sea surface for all stages of the tides for an 18.6 year period. This period is required because the sun and moon, which affect the tides, repeat the same pattern every 18.6 years. The height of mean sea level is determined from continuous measurements made with automatic tide gauges set in relatively calm water. Gauging stations have been established and are maintained by NGS at regular intervals along the coast and along tidal rivers. NGVD 29 was computed based on twenty-one tidal stations in the US and five in Canada.

These gauges are connected through tidal bench marks to a precise network which covers the 48 contiguous states. When the network was run, the height of mean sea level was found to vary slightly from one tidal station to another. In 1929 the network was adjusted so the elevation of mean sea level at each gauge was zero. This established the 1929 Sea Level Datum (SLD). In 1973 the name of this datum was changed to National Geodetic Vertical Datum of 1929 (NGVD 29).

Similarly to the experiences with the horizontal datum which was discussed earlier, NGVD 29 was found inadequate for modern day surveying. It became necessary to redefine and readjust the entire system of vertical control or bench marks.

2.1.3.2 The North American Vertical Datum (NAVD 88)

In 1991 the National Geodetic Survey completed a general adjustment of the North American Vertical Datum of 1988 (NAVD 88). NAVD 88 supersedes NGVD 29, which was the height reference for the United States. NAVD 88 provides a modern, improved vertical datum for the United States, Canada and Mexico. NAVD 88 heights are a result of a mathematical least squares general adjustment of the vertical control portion of the National Geodetic Reference System (NGRS) and include 80,000 kilometers of new U. S. leveling observations undertaken specifically for this project.

Extreme care must be used to insure that all height values of individual bench marks used within a project are referenced to only one vertical datum. In the State of New Jersey the difference between NGVD 29 heights and NAVD 88 heights varies between 0.18 to 0.4 meters (0.6 to 1.22 feet). Therefore, the mixing of these datums could cause many errors in the vertical portion of a survey.

2.1.3.3 Local Vertical Datums

During the original level surveys through the State, bench marks were established in every city or town. The bench marks were generally located near the courthouse, railroad depot, or other prominent building within the town limits. Most of the towns and cities have extended that control by "benching" fire hydrants or other semipermanent points. Generally, throughout a municipality, the control is fairly consistent. Due to the 1929 adjustment and subsequent refinements of the USC&GS network, most of those local datums do not agree with current network elevations.

In order to maintain a consistent datum throughout highway projects, extreme care should be used to identify which benchmark and which datum were referenced. Benchmarks must be referenced to the NGVD 29, NAVD 88, assumed or local datum. If this information is known, it is then a fairly simple procedure to establish a vertical equation for local systems by leveling between the local and NGVD 29 or NAVD 88 benchmarks.

2.1.4 Plane Coordinate Systems (General)

A coordinate system is used to determine the relative position of points within the survey area or, in many cases, with respect to a much larger area. In order to make the coordinate system usable for engineering projects, the horizontal relationships should be defined as two dimensional on one (mapping) plane. To make the coordinate system usable and to simplify linear measurement, the coordinate system should be rectangular so that equal values measured from a datum axis form a parallel line with that axis. Parallel lines to each of the two axes form a “grid" and the intersection of those lines are rectangular "grid coordinates”. This type of coordinates are called Cartesian coordinate systems.

The following table summarizes the differences between plane and ellipsoidal coordinates

	Plane	**Ellipsoid**
North-South Direction	Straight up or parallel to the direction of the North Arrow	Slanted (not uniformly) towards the North Pole. All lines pointing to North converge at the North Pole.
Distances	Straight lines	Curved lines
Sum of angles in a quadrilateral	360°	360° + spherical excess
Even coordinate differences correspond to:	Even (same length) distances.	Uneven distances, i.e. the length of an arc of 2° of longitude near the pole is much shorter than 2° of arc near the equator.

Table 1. Some differences between working on a plane and on an ellipsoid.

In highway work, the north-south axis is designated as the Y-axis and the east-west axis as the X-axis. The horizontal distance from the Y-axis is the "easting" coordinate and the vertical distance from the X-axis is the "northing" coordinate. A statement of the exact position of a point within the system can be expressed as X-Y, or easting-northing value of the point. Stated either way, the point location is being described by rectangular coordinates and the location is a grid location.

Another statement of the exact position of a point (P) within the system can be expressed as a direction and distance from the origin (O, intersection of the Y and X axes). In this case, the location of the point is being described by polar coordinates and the location is the polar location.

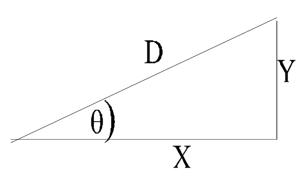

Figure 2-1. Rectangular coordinates (X,Y) vs. Polar coordinates (D,θ)

Due to the rectangular nature of the grid system, the conversion between rectangular and polar coordinates is accomplished by simple right triangle plane trigonometric calculations. The conversion is used throughout the survey. For instance, a traverse is started at a grid location described by the X and Y coordinates. The direction of the first leg of the traverse is established by turning an angle from a known direction to the next traverse point (thus establishing the polar direction); the distance is measured between the points (thus establishing the polar distance). A simple right triangle solution adding the northing and easting of the line to the northing and easting of the beginning point to give the rectangular coordinates of the new traverse point. Conversely, a point with pre-determined X,Y coordinates may be established from a pair of known points by calculating the polar relationships between the point occupied and the point to be established. A predetermined angle is turned to the calculated polar direction and the point set at the calculated polar distance. This simple operation, called radial surveys, is extremely useful in setting Right-of-Way markers, slope stakes or other construction control points from project control points.

It is noteworthy to mention here that in order to make volumetric calculations, set grade lines and establish vertical clearances, etc., the third dimension (elevation) is shown on the plane map surface as contour lines (a line depicting equal elevation) or spot elevations. A combination of horizontal position and elevation data makes up a "topographic map". This same data in digital form is called a Digital Elevation Model or (DEM.)

To define a plane coordinate system one has to select an origin for the X-Y coordinates and a direction of the Y-axis or the north. The selection of these coordinate system parameters can be done on a local, state or even on a worldwide basis.

2.1.4.1 Assumed (Local) Plane Coordinate System

Traditionally, most surveys used an assumed or local plane coordinate system. A point is assigned an arbitrary coordinate, such as X=10,000 and Y=20,000, and a direction from that point to a nearby point is set to be due north. In the case of a closed traverse (which is the most widely used surveying operation) the first point is assigned the arbitrary coordinates and the direction from the first to the second point is fixed as due north. An assumed plane coordinate system has two major drawbacks.

1. It does not accommodate splitting a large project into smaller independent projects. Dividing a project into smaller ones is desirable in circumstances of time and/or budget constraints.

2. The approximation of the surface of the earth to a plane is valid for only a limited extent of an area. Beyond that, the corrections have to be applied to distances and angle to reflect the deviation of the curved earth from a plane.

For small and isolated projects, such as local property surveys or construction surveys, an assumed plane coordinate system can be acceptable. However, for large engineering projects, such as a lengthy highway, this practice must be avoided.

2.1.4.2 State Plane Coordinate System

It is impossible to map a curved Earth on a flat map using plane coordinates (X,Y or northing, easting) without distorting angles, distances or areas. However, it is possible to design a map projection such that some of these elements are undisturbed or minimally distorted. The State Plane Coordinate System (SPCS) is a map projection system that minimizes angular distortions if only a small portion of the Earth is flattened out. The (X,Y) coordinates are computed by projecting latitudes and longitudes from a mathematical approximation of the earth (i.e. NAD 83) onto a surface that can laid out flat. The three surfaces that are used for this purpose are a plane, a cone or a cylinder. The earth is wrapped with a cone or a cylinder and the earth's features are projected onto it. When the projection is done the cone or the cylinder are cut and opened into a flat surface.

SPCS consists of a set of mathematical relationships that are used to convert northing and eastings into latitude and longitudes and vice versa. It also includes a set of formulas to compute the size and the direction of location displacement (positional error) resulting from the projection process.

In the US there are mainly two projection systems utilized for the state plane coordinate system. The first is the Transverse Mercator (TM) which best serves states (or portions of states) which extend in the north-south direction. The other projection system is the Lambert Conformal Conic (LCC) projection that is best suited for mapping long distances in the east-west directions. The TM projection utilizes a cylinder, resting on its side, for projecting points from an ellipsoid into a plane. Thus, along one meridian (i.e. north-south) there is no distortion at all regardless of the latitude. The LCC projection utilizes a cone so that along a given parallel (i.e. east-west) there is no distortion regardless of the longitude.

2.1.4.3 Benefits and Need for Use of the State Plane

1. All surveys correlate to a single reference framework. This means that all surveys, old and new, can be combined seamlessly into a consistent and contiguous mapping project. Points from old and new surveys can be used without the need to re-calculate the old measurements. Surveyors having numerous projects in a certain area could, theoretically, "cut and paste" different projects to produce a map without compromising the accuracy of the new product.

2. Large projects can be surveyed in parallel as independent sections. Although during the time of the execution of the project the different sections are not yet connected physically, they are connected computationally because they all share a common

reference framework. As the work progresses, all sections will be connected and the accuracy of the entire project will be maintained throughout.

3. Data sharing among surveyors is simplified if everyone is working on the same reference system. Data is a precious commodity in the GIS/LIS world. Surveyors have an abundance of spatial information. If it is in a useable form (such as SPCS), it has a market value.

4. No point can be considered lost because it can be recovered from its coordinates. For example, if a point has State Plane Coordinates, it can be reestablished by using GPS. There is no need to recover points from ties (which may have also been destroyed), unless there is a legal issue involved.

5. Using SPCS, the earth can be viewed mathematically as a plane. This means that plane geometry and trigonometry mathematics can be used in the computations. One needs only to apply a small, well defined, correction to compensate for the plane approximation. This manual explains what corrections have to be made.

6. Working with SPCS provides an extra external computation check for the surveys. Loop closures, such as a closed traverse, check only the inner consistency of the survey. If, for example, there is a systematic scale error in the traverse, it will not be detected by summing up the latitudes and the departures. Only when the traverse is connected to two or more points with given State Plane Coordinate values can this error be discovered and corrected. A similar argument holds for the orientation of the traverse. To maintain proper orientation of a traverse, it has to be connected to at least two control points with State Plane Coordinates.

7. Use of the system of State Plane Coordinates is vital to accomplishing precision mapping, highway design and location, and cadastral surveying on a statewide basis.

8. By law, the New Jersey State Plane Coordinate System based on NAD 83, is the official survey base for the State of New Jersey (N.J.S.A.51:3-7).
9. Use of SPCS accommodates statewide GIS/LIS activities.

2.2 New Jersey State Plane Coordinate System

The New Jersey State Plane Coordinate System is based on a Transverse Mercator (TM) projection. The reason for using the TM projection is because the shape of New Jersey is elongated in a north-south direction. As mentioned earlier, projection distortions in a TM projection increase as the east-west distance from the central meridian increases, but remain rather constant in the north-south direction. The best projection for a state is the one that introduces the least distortions.

There are two New Jersey state plane coordinate systems. The first is based on NAD 27 and the other is based on NAD 83. The NAD 27 SPCS is the earlier one and should be avoided. According to N.J.S.A. 51:3-7, the official state plane coordinate system for New Jersey is the one based on NAD 83. State plane coordinate values of NAD 83 are different from those of NAD 27 because:

- Change in datum (NAD 83 is based on GRS 80, NAD 27 is based on Clarke 1866)
- Changes as a result of a new adjustment
- New mapping (projection) equations (derived mathematically not empirically) to support 1mm accuracy
- Changes in numerical grid value of the origin of each zone
- Changes in mapping constants in some zones (new standard parallel or meridian)
- Azimuth orientation is due North
- The use of Metric units rather than English units.

The parameters of the New Jersey state plane coordinate systems are:

SPCS	NAD 27	NAD 83
Zone (NGS) Code	2900	2900
Projection	TM	TM
Scale Factor	1:40,000	1:10,000
Origin latitude: φ	38° 50’	38° 50’
Central Meridian: λ	74° 40’	74° 30’
(X) E at Central Meridian	2,000,000	150,000
(Y) N at origin latitude	0	0
Units	Feet	Meters
Initial Azimuth due	South	North

2.2.1 The Surveyor and SPCS

A surveyor should have a basic understanding of the derivation of this grid system and the relationship of the various components of the Transverse Mercator Projection. In most cases, inverses computed from grid coordinates do not correspond to measured values in the field. The surveyor must understand why the discrepancy exists and properly apply the necessary corrections so that field measurements and coordinate geometry computations are consistent. Thus, it is important to understand the relationship between a point on the topography and its representation on the state plane coordinate system.

2.2.2 From Topography (Surface) to State Plane

In the geodetic reference system, coordinates of points (latitudes and longitudes) and the lengths and azimuths of lines are defined on an ellipsoid. Therefore, surveys that are to be adjusted to stations of the national control network must first be reduced to the ellipsoid. Since the state plane coordinate systems are developed directly from geodetic values, the use of those systems require the further reduction of the ellipsoid values to grid values. The reduction from ground to the state plane is a simple two-stage process. Reduction from ground to the ellipsoid is called the "elevation factor" and reduction from the ellipsoid to the state plane grid is called the "scale factor”. The scale factor in New Jersey is set to 0.9999 at the central meridian and it increases as a function of easterly or westerly distance of the point from the central meridian. The maximum value of the scale factor is 1.0001.

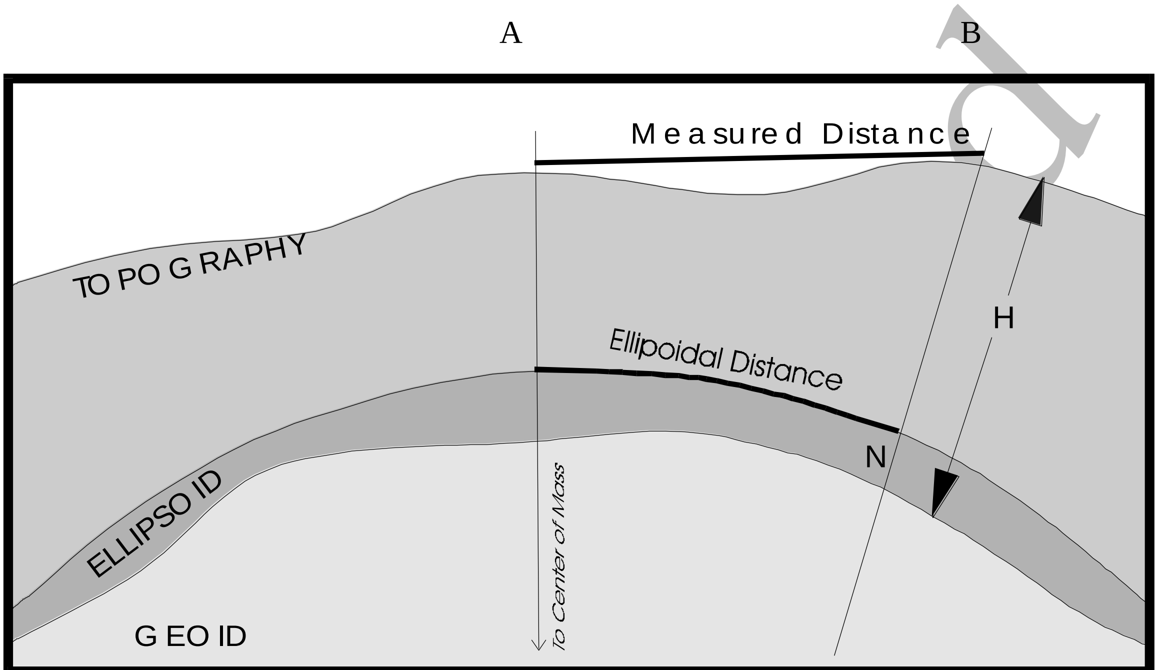

Figure 2-2. The relationship between a measured distance and a distance on the ellipsoid.

The following sections contain the formulas for reducing a distance from the topography to the state plane grid.

2.2.2.1 Reduction I. (Grid) Scale Factor K_{AB} for a Line from Point A to Point B

Purpose: To correct for scale distortion due to the projection of the ellipsoid onto a plane.

Formula: $$K_{AB} = \frac{K_A + 4K_m + K_B}{6}$$

Where:

- K_{AB} - Grid scale factor of a line between points A and B.
- K_A - Grid scale factor at point A
- K_B - Grid scale factor at point B
- K_m - Grid scale factor at the line's mid- point.

Usage: A reasonable approximation for the above formula is to compute a simple average of K_A and K_B . A further approximation is to compute a single K value for the entire line or for the entire survey area.

For New Jersey, the value of K can computed for a point with a given easting, E, from:

$$K = 0.9999 + (E - 150000)^2 \cdot 1.23 \cdot 10^{-14}$$

2.2.2.2 Reduction II. Elevation Factor, Reducing Measured Horizontal Distance to Grid Distance.

Purpose: To reduce a measured horizontal distance to the projection plane. As mentioned earlier, field measurements are carried out on the physical surface of the earth, while office computation are performed on the projection of the earth onto a plane. This reduction is in essence the bridge between field measurements and the computations on the state plane coordinate system.

Formula:

$$S = D \times \left(\frac{R}{R + H + N}\right) \times K_{AB}$$

Where:

S	-	Grid Distance
D	-	Horizontal (Measured) Distance
H	-	Mean Elevation (Above Mean Sea Level)
N	-	Mean Geoid Height (About -32m or –100ft in NJ)
R	-	Mean Radius of the Earth (About 6,372,000m or 20,906,000)
K_{AB}	-	Grid Scale factor of the Line.

Usage: Obtain the elevations of the terminal points of the line, the Geoid height of the region, and calculate.

Although the above discussion appears complicated, the solution for surface measurements to state plane distances are quite simple to program on programmable calculators or computers.

2.2.3 State Plane to Surface Coordinates

The surveyor's primary concern is with surface measurements and the use of the coordinate relationship between points established, or to be established, on the ground. To compute a distance 'D' between two points on the ground from their state plane coordinates (grid) distance 'S', the above corrections must be applied in a reversed order. The inverse correction is:

$$D = S \times (\frac{R + H + N}{R \times K_{AB}})$$

In some situations it is impractical to adjust every surface measurement to determine the state plane coordinates of the point. There are some cases where it may be advantageous to produce engineering mapping on a plane that closely relate to the ground on which that facility is being designed and constructed.

The advantage of having a coordinate system that can be as closely related to the ground as practicable is obvious. The danger is that this data could erroneously be confused with state plane based surveys. Therefore, the coordinate listings should be clearly identifiable so that surface coordinates cannot be confused with state plane grid coordinates. Other criteria for establishing a coordinate system (Project Specific Adjusted State Plane Coordinate) for engineering projects should include:

a. Adjustment between ellipsoid and actual ground measurement should be minimal.

b. Coordinate relationship between the surface grid and the reference state plane should be constant and simple to calculate.

c. The surface coordinate system should extend throughout the project area so that no coordinate equations are required. If practical, the coordinate system should be made compatible with adjacent projects.

In accordance with the above criteria, surface coordinates should be established for the project, which best fit the elevation of the ground. A Project Adjustment Factor (PAF) should be calculated for each project using the average of the combined factors for reducing surface distance to state plane distances. Some NJDOT mapping and engineering calculations are produced on the surface plane and listings indicate surface coordinates.

2.2.4 Universal Transverse Mercator (UTM)

The Universal Transverse Mercator (UTM) system is another important plane coordinate system. Originally developed by the military for artillery use, it provides worldwide coverage from 80° South latitude to 84° North latitude. Each zone has a width of 6° 00'00", which requires 60 zones to cover the entire earth. The zones are numbered easterly from 180° West longitude. New Jersey is located in zone 18.

Longitudes and latitudes used in UTM calculations are based on Clarke 1866, the same ellipsoid used in NAD 27.

The UTM system is a modified transverse mercator projection. It has recently taken on new importance since UTM coordinates are widely used in GIS/LIS. Current USGS 7 1/2 minute quad sheets, as well as other maps in the national mapping program now show 10,000 meter grid ticks to allow manual plotting in this system.

The central meridian of each zone is assigned an easting of 500,000 meters and a northing of 0 meters is applied to the equator for the northern portion and 10,000,000 meters applied to the equator for the southern portion. The scale factor at the central meridian is set to 0.9996 or a relative accuracy of 1:2500. UTM was not adopted by NGS as the state's plane coordinate system for several reasons. One reason is that a relative accuracy of 1:2500 is too low to make it practical for surveying projects. Another reason is that UTM does not conform to state boundaries, which is a desirable feature for a state plane coordinate system. It is desirable to place the central meridian or the standard parallels in the center of a state to minimize projection distortions. This cannot be done with the rigid structure of the UTM system. Surveyors may encounter UTM when dealing with buildings on the National Historic Register, sewer outfalls, historic sites, etc.

2.2.5 Azimuths

2.2.5.1 Definition

Azimuth is the horizontal direction of a line measured to the right (clockwise) from a reference meridian. In the past (NAD 27), azimuths published by the various government survey branches were measured from the south. Since the development of (NAD 83), new data for all NGS control points are published with zero azimuth pointing to the north.

It is noteworthy to mention here that most calculators and computers use the mathematical convention for reference (zero) azimuth which points towards the east and measures counterclockwise. Angles in computers are also usually expressed in radians. Surveyors must also be alert and realize that some agencies also use grads (1/400 of a circle) for the angular definition. Azimuths in grads should not be confused with those expressed in degrees of arc (360°). These various definitions of azimuths are generally easy to overcome when known, but the surveyor should be alert to recognize the differences and know how to make the appropriate change to the system he or she is using.

NJDOT, local and other state agencies and private surveyors are north oriented. The advantage of north oriented azimuth is that the algebraic signs of trigonometric functions are correct when calculating latitudes and departures (in traverse computation).

There are three basic azimuth systems used in highway surveys:

A. Astronomic azimuth is the angle between the (instantaneous) true north and a vertical plane through the observed object. Astronomic azimuths are usually measured to maintain proper orientation of long traverses along a highway. Astronomical azimuths are determined from observations to the Sun or to Polaris. The use of GPS has diminished the utilization of astronomic azimuths in route surveys.

B. Geodetic azimuth is the angle between the meridian of the ellipsoid representing the earth and a plane perpendicular to the ellipsoid through the observed object.

C. Grid azimuth is the angle in the plane of the projection between grid north and the straight line from the point of observation to the point observed. Grid azimuth is the same as geodetic azimuth only when the point of observation falls on the central meridian.

2.2.5.2 Forward and Back Azimuths

The azimuth for a given line, AB, is usually stated as the azimuth measured at point A towards point B. This is called the forward azimuth. However, each line has a corresponding back azimuth, BA, which is the azimuth at point B towards point A. For grid azimuths, the difference between forward and back azimuths is always exactly 180°. This is not the case with geodetic azimuths. Because of convergence of the meridians, the difference between forward and back geodetic azimuth is 180 degrees plus the difference in the angle of convergence and a small arc-to-chord correction.

2.2.5.3 Conversion From Grid Azimuth to Geodetic Azimuth

It is useful for the surveyor to know the difference between the geodetic and grid azimuths of a survey line for the purpose of obtaining a check on a computed value or to provide a starting azimuth for the survey. It may also be useful to convert grid azimuth to geodetic azimuths of property lines to an azimuth that is independent of the choice of map projection. For this, and other reasons, the issue of convergence angle is important to surveying.

Relationship Between Geodetic and Grid Azimuths

Purpose: To account for the convergence of the north direction towards the pole vs. parallel north direction on a plane.

Formula: $\boxed{AZ_{Grid} = AZ_{Geodetic} - \gamma + (t-T)}$

Where:

AZ_{Grid}	-	Grid Azimuth
$AZ_{Geodetic}$	-	Geodetic Azimuth
γ	-	Meridian Convergence
$(t-T)$	-	Arc-to-chord correction

Usage: This formula (except for the t-T correction) is used only when it is necessary to convert grid azimuth to geodetic azimuth, or geodetic azimuth to grid azimuth. Grid Azimuth can be computed from an inverse between two points with plane coordinates. Geodetic Azimuth is usually provided by NGS or from GPS measurements. It can also be computed from Astronomical observations corrected for the deflection of the vertical (called Laplace Correction).

The value of the meridian convergence at stations of the national control network is listed on their data sheets published by NGS. There are software and interpolation tables to compute this convergence angle. The magnitude of the convergence angle in NAD 83 is illustrated in figure 2-3:

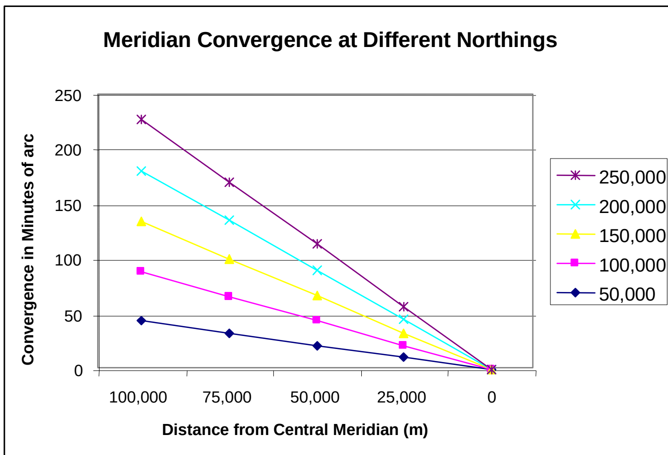

Figure 2-3. The meridian convergence as a function of the distance from the central meridian

Arc-to-Chord Correction (t-T) for line 1-2.

Purpose: The (t-T) correction is due to the fact that the measured direction between two points is actually a curved line, on the surface on a body such as an ellipsoid, that passes through these points. When projected onto a plane, the geodetic direction looks like an arc, rather than a straight line. The angle that is computed from field notes is defined by the difference between two measured directions. Thus, the computed angle differs slightly from the plane angle of the state plane coordinate system. This difference is expressed by (t-T).

Formula: $\boxed{(t\text{-}T)'' = 25.4 \times \Delta N \times \Delta E \times 10^{-10}}$

Where:

$\Delta N \quad = \quad N_2 - N_1$

$\Delta E \quad = \quad \frac{E_2 - E_1}{2} - E_0$

$(t\text{-}T)''$ - Arc-to-chord correction in seconds of arc.

N_1 - Northing of point 1

N_2	-	Northing of point 2
E_1	-	Easting of point 1
E_2	-	Easting of point 2

Usage: The size of the correction is rather small and can be neglected for most ordinary work (not high accuracy). The sign of the correction is dependent on the direction of the line with respect to the north (Azimuth dependent). The magnitude of the correction in New Jersey is:

	Average Distance of the line from the Central Meridian (m)			
ΔN	150,000	100,000	50,000	0
2 km	0.8"	0.5"	0.3"	0"
5 km	1.9"	1.3"	0.6"	0"
10 km	3.8"	2.5"	1.3"	0"
20 km	7.6"	5.1"	2.5"	0"

Where ΔN is the difference in Northing between the end points of the line.

2.3 Geodetic Control

2.3.1 Introduction

The Federal Geodetic Control Committee (FGCC) and subsequently the Federal Geographic Data Committee (FGDC) have developed standards and specifications for establishing horizontal and vertical control. The (NGS) has developed a standard format for geodetic survey data, commonly known as "Bluebook" format. This format must be used if geodetic survey data is to be submitted to the NGS for review and inclusion into the National Network.

It must be noted that the FGCC Standards and Specifications also require a survey control monument to meet certain physical conditions. It is not reasonable to require that a survey meet first-order requirements and submit data to the NGS if the monuments do not meet the same standards. Not every point on every survey project or a subdivision has to be included in the national geodetic network of control points. It may be reasonable to set a few monuments throughout a project area that meets federal monumentation standards. These points are to be submitted as primary control stations. The primary control stations in this example would be "bluebooked" and included in the national network. The remaining points are to be monumented with lower and less expensive standards, and referenced to the primary monuments.

The recommended procedure is to use (GPS) survey methods to establish horizontal control, use spirit leveling methods for establishing vertical control when elevations of highest accuracy are needed, otherwise elevations determined by GPS may be adequate for most projects. The surveyor should follow FGCC Standards and Specifications for the surveying of all geodetic control. He/she should also 'Bluebook" the primary, if not the entire, survey control project and submit it to the NGS for review and inclusion into the national network.

2.3.2 Survey Control Network

To facilitate the creation of GIS/LIS, all spatial objects must be maintained in a common coordinate system. This means that they have to be tied to control points with geodetic or state plane coordinates. Since the primary geodetic control is too sparse, there is a need to densify the survey control network. Arbitrarily setting monuments as needed for the mapping/GIS/LIS control is one pattern that can be used for physically placing survey control throughout the project. Another pattern would be some systematic planning of control points that are easily accessible for a variety of projects.

The survey control network is the framework of the entire project. It will only be as accurate as the control survey upon which it is based. The cost of the survey control network is a small percentage of the overall cost, ranging from 1% to 5% of the total survey cost being invested in the project. Factors affecting cost include:

A. Setting new monuments or occupying existing monuments (property corners, road centerline monuments, etc.). Being able to incorporate existing monumentation, when available would save not only the cost and time of physically setting a new monument, but would also allow the project to have actual ties to property line and road right-of-way information.

B. Number of survey control monuments being established for the project. Mobilization for performing the survey can affect the cost in several ways. Fewer monuments will make the cost of mobilization higher per monument, and fewer monuments also means more time is spent traveling between survey control stations, causing higher costs per station for performing the survey. A greater number of monuments will decrease the cost per station for both mobilization and performing the survey.

C. Terrain and location can affect the cost of performing the survey. Tree cover, urban areas and hilly areas can also make the planning and implementation difficult. Road occupations in busy intersections can incur additional costs for traffic control.

D. "Bluebooking": Survey control submitted to NGS in their format will have an extra quality check by the federal government at no charge, but there is a cost incurred in the time spent formatting the data to "bluebook" standards.

E. Time spent to go farther away from the project area to recover needed horizontal and vertical control, if there isn't enough available control in the project area, increases the cost of the survey.

2.3.3 Planning

It is important to plan for the future use of the control points from the beginning stages of developing the project. The geodetic survey control framework is the foundation of the project. If proper consideration is given to the quality and accuracy needed for current and future needs, the project will be much easier to use and to keep updated in the maintenance/update phase. Surveys for new highways and engineering projects, etc., can be readily incorporated into a GIS/LIS if they are accurately referenced to the control network. All personnel involved in geodetic surveys should become familiar with NGS bluebooking procedures before beginning work on a project. See the Regional Survey Services Supervisor for these procedures.

2.3.4 Conclusions

Do not cut costs. The survey is the framework of the entire project and a small percentage of the overall investment. Establish horizontal coordinates on each monument by following "bluebooking" and other specifications. Plan for the use of the survey control in the future, both to be able to accurately update the transportation network and for future requirements for the surveyors to tie into GIS/LIS. This planning includes establishing "station pairs", performing the geodetic control survey to FGCC specifications for second order accuracy, or higher, and setting permanent survey monuments.

2.4 Monumentation

2.4.1 Definition

A monument is a fixed object on the ground, whether natural or artificial, serving as a permanent mark of the survey. The monumentation of control survey points is an important part of a highway project. Hundreds of thousands of dollars in surveying work have been lost because surveys were not properly monumented or the monuments were destroyed. A control survey monument should possess stability, positive identity and protection against destruction. The practice of monumenting control points should be continued even though GPS has made it easier to reestablish the location of points with known coordinates. As long as surveys are performed with equipment other than GPS, monumentation remains indispensable. If the control point is associated with property records, its monumentation is important from the legal boundary recovery perspective.

2.4.2 Purpose

It is apparent that time spent on monumentation serves a three-fold purpose:

A. The monuments provide a permanent physical object that is readily acceptable by local surveyors as the survey control for the roadway.

B. The monuments can be incorporated into a city, county or state system to enhance the control network GIS/LIS.

C. The monuments can be incorporated into the National Geodetic Reference System (NGRS) and referenced to that system.

2.4.3 Characteristics of Monuments

With the increasing use of GPS measurement techniques which rely on relative positioning, it is important that station markers have the properties of permanence and stability. The markers must be stable in all three dimensions.

Factors that may affect the stability of a monument include frost heave action, changes in ground water level, and settlement. When selecting sites for stations of a high precision primary network or for monitoring deformation, it is recommended that soil and geothermal specialists be consulted.

Markers for existing network control should show no historical evidence of significant movement. If an existing network control marker does not adequately exhibit the properties of permanence and stability, it may have to be replaced by a new marker. The decision to replace old markers will depend on their use and purpose in future surveys.

To meet the requirements for permanent and stable monumentation, the markers are usually bronze disks that may be set in rock outcrops or large masses of concrete such as bridge abutments and other structural foundations.

When bedrock or large massive structures are not available, it is more difficult to ensure the monument has the properties of permanence and stability. Traditional concrete monuments, with or without an underground mark, are not recommended as a suitable choice for preserving the three dimensional coordinates. A three dimensional (3D) driven survey monument will provide the necessary stability for most conditions. Reference the NJDOT Minimum Guidelines for Aerial Photogrammetric Mapping Manual page 4-32 for details.

2.4.4 Monument Types

There are several types of monuments in general use. They vary from 3D-driven rod monuments, cast in place concrete monuments and monument assemblies for permanent monumentation to iron pins with caps for semi-permanent applications. The monument used depends on the type of survey, local conditions and the requirements of the project. In highway engineering there are mainly four types of monuments.

A. 3D - driven rod monuments. This monument is to be used for points that require maximum stability such as (HARN) stations. Follow NGS specifications for this type of monuments.

B. Standard tablets set in concrete, rock or existing structures. This type of monument is used mainly for marking primary survey control points, (i.e. GPS, triangulation, trilateration, and primary traverse).

C. Survey capped iron pins. This monument, if properly referenced and witnessed is probably the best for marking the stations of a supplementary control traverse. Iron pins are also used as right-of-way monuments.

D. Iron pin with cap set in concrete. Rock outcrops or existing concrete structures, such as bridges or culvert headwalls, may be used for setting permanent marks. Bronze or aluminum tablets firmly implanted in concrete are as permanent as the structure in which they are placed. These monuments can be used for marking any kind of survey control. When properly set in concrete, the standard monument is relatively indestructible.

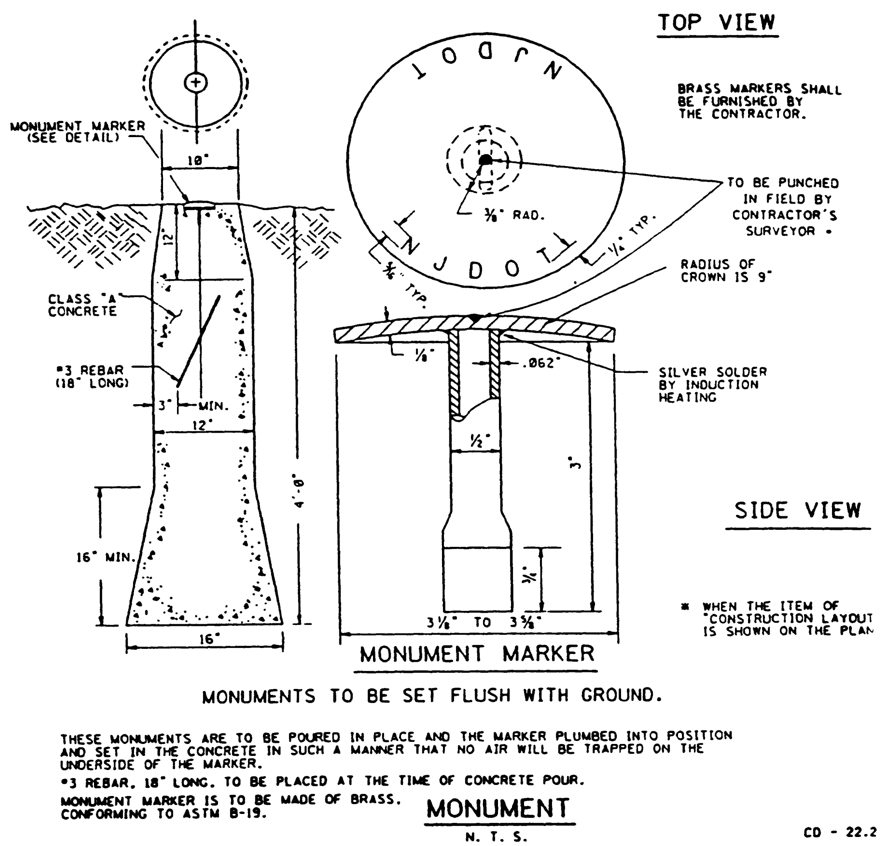

Figure 2-4 New Jersey DOT monument

For detailed construction of these monuments, see Figure 2-4 and the NJDOT standard construction drawings.

2.4.5 Cost

The cost of monumentation relatively speaking is small compared to the millions of dollars routinely spent on highway projects. The cost involved in monumentation is far outweighed by the benefits derived, and an effort should be made to monument existing highways**.** Whether the project is a new road or a major renovation every effort should be made to include the cost of monumentation in the plans.

2.4.6 Referencing Monuments

A written description and a location sketch of each monument should be made in the field. Monuments must be tied to either permanent natural objects or to additional points set nearby**,** including angular references to visible landmarks.

A steel or fiberglass witness post should be set near each 3D - driven rod monument and existing concrete structures. These posts should provided with a warning, 'DO NOT DESTROY PROPERTY STATE OF NEW JERSEY,' to serve as a warning to others and make a significant reduction in the number of monuments destroyed.

2.4.7 Right-of-Way

Right-of-way monumentation is required on all new property acquisitions by NJDOT (see Appendix A). Right-of-way monuments shall be set by NJDOT and NJDOT consultants at the time of staking the bounds of fee parcels and permanent easement parcels acquired by NJDOT, and shall be called for in the legal descriptions. Monuments will be set along NJDOT ROW, or baselines at PC's, PT's, POT's and will be stamped "NJDOT R/W" when set by NJDOT personnel. Monuments set by consultants, will be stamped "NJDOT R/W' and company name in accordance with NJAC 13:40-5.1(d)3. Also, reference section 6.3.3.5.