بسم الله الرحمن الرحيم

Palestine Polytechnic University

College of Engineering and Technology
Civil & Architecture engineering Department

Graduation Project

Spatial Data Quality And Map Assessment

Project Team

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Hebron – Palestine

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الإهداء

Dedication

إلى من رووا بدمائهم الزكية تراب هذا الوطن الغالي

إلى الذين حملوا أرواحهم على اكفهم من اجل وطنهم

إلى والدتي و والدي و اخوتي... نبع الحنان و العطاء

إلى الأصدقاء... نبع الحب و السعادة

إلى رواد العلم المبدع، طلاب اليوم، بناة المستقبل، قادة الغد

إلى طلاب جامعة بوليتكنيك فلسطين من اخوتنا وأخواتنا... من خالفنا منهم الرأي أو من اتفق معنا وانطلق لآفاق التجدد و الإبداع.

فريق العمل :

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WORK TEAM

ABSTRACT

Spatial Data Quality and Map Assessment

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Spatial data quality and map assessment is a project that search for accuracy positions in Hebron city municipal topographic map, which produced using Photogrammetry techniques.

To make assessment for this map we adopt standards and specifications according to National Map Accuracy Standards *(NMAS)*, United State Geological Survey *(USGS)*, and American Society for Photogrammetry and Remote Sensing (*ASPRS, 1989*), Federal Geographic Data Committee *(FGDC, 1998).*

A traverse used for determining the positions of checkpoints to make assessment, and then make a reduction for distances to mean sea level surface and Azimuth bearing (t-t correction) for angles to make precise traverses.

Accuracy assessment of the Hebron city municipal topographic map has been tested by four agencies, in which the map passed the accuracy assessment.

جودة المعلومات و دقة الخرائط

إعداد

حسام محمد الجعافرة **فادي فايز مصطفى**

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إشراف

المهندس كمال غطاشة **المهندس فيضي شبانة**

:

جودة المعلومات ودقة الخرائط: هو عبارة عن مشروع يبحث دقة المواقع في الخرائط التي يتم إنتاجها عن طريق تقنية التصوير الجوي

لعمل تقييم لهذه الخرائط تم تبني معايير و مواصفات لمؤسسات *(NMAS)*، *(USGS)* ، *(ASPRS, 1989)*، *(FGDC, 1998)*.

تم اختيار المضلع المغلق في العمل المساحي لتحديد مواقع النقاط المراد عمل التقييم لها، و تم إدخال تصحيحي (reduction) للمسافات (انزال المسافة الى مستوى سطح البحر) و (Azimuth bearing) للزوايا، وذلك للحصول على المضلع الدقيق.

تم في هذا المشروع تقييم خارطة طبوغرافية لمدينة الخليل، تم الاختبار بالاعتماد على أربع مؤسسات عالمية ونجحت هذه الخارطة وفقا لمعايير و مواصفات هذه المؤسسات.

Chapter One

1

INTRODUCTION

CHAPTER 1

INTRUDUCTION

1.1 INTRODUCTION

Data quality is one of the critical problems facing organizations today, as the management and decision–making become more information dependent. Poor data quality is pervasive and costly.

Quality has various definitions in engineering, but the acceptable definition is the “fitness of use”. Others consider even this definition as a data requirement. From above it is clear that quality is linked with error. [Source: Reference No.3]

However, error in maps and databases creep into data through primary and secondary methods of data collection. Primary methods refers to the methods where data are collected directly from the field e.g. Photogrammetry, land surveying, and satellite imagery, while secondary methods refer to data collection from the existing hard copies.

Maps and databases are assessed by different approaches depending on the type of error to be measured. Thematic errors for example are measured by the construction of what is known as confusion matrix. Error in positional accuracy can be assessed by the use of a reference layers or by the use of a group of checkpoints.

In spatial data, the quality is linked to error and the later is expressed by different criterions or standard, such as error ellipses, relative error, and circular error.

If any, little attention is paid to spatial data quality issue in the Palestinian institutes, a situation that causes a lot of national losses in money terms. This project gives an overview on some of the prevailing international standards, specifications, and methodologies used in spatial data assessment. We are sure that stressing the data quality issue may encourage the Palestinian institutes to adopt, or upgrade the necessary standards and specifications to control the data quality of their maps and databases according to the intended use.

1.2 PROJECT IMPORTANCE

1. The project raises the map quality issue and stimulates the Palestinian institutes to give it more attention.
2. The project shows the importance of adopting certain standards and specifications.
3. The project shows different techniques used to assess the map quality and spatial databases products.

1.3 PROJECT OBJECTIVES

The main objectives of the project include the followings:

1. To draw the light on the significance of standards and specifications of spatial data.
2. To disseminate awareness on the data quality issue for maps and databases in the Palestinian community.
3. To assess the situation of spatial data quality issue adopted in the working institutes in the Palestinian community.
4. To test one of the existing maps, against certain standards, as an example on map testing approach.

1.4 STUDY CASE

A topographic map, for the city of Hebron, compiled by photogrammetry technique was used as a study case in this project. This map shows the roads, building, and contour lines. Unfortunately no Metadata is attached with this map.

1.5 METHODOLOGY OF WORK FOR THE CASE STUDY

1. Select a map to assess, a topographic map for the city of Hebron is chosen. The map has no Metadata. The map shows details of buildings, road boundaries, and contour lines with 2.5m contour intervals. The map was compiled by photogrammetric techniques.
2. Divide the map to four equal quadrants.
3. Select equal number, as a possible, of well-defined points in each quadrant.
4. Determine the reference checkpoints location, a suggested closed traverse to pass very near to the selected points on the map as possible.
5. Determine the quality of checkpoints by error propagation techniques. These checkpoints should match the Federal Geodetic Control Committee *(FGCC)* standards of control points.

We couldn't b able to design a survey work, which meets a high standard of accuracy due to the following reasons:

- Non-availability of very precise instruments.
- The absence of the instrument manual (which states the probable precision of instrument) and the absence calibration of the instrument.

Therefore, we are going to use the available instrument and make some assumption regarding:

1. Angular precision (standard deviation) in angle measurement$\pm\sim$. This will include pointing, reading, and centering errors.
2. Systematic error corrections for distance measurements cannot be carried out due to the non-availability of pressure measuring equipment. Therefore, we are going

to assume $\pm d$ (standard deviation in distance) for the EDM, without the correction of systematic error.

- So, at the end of our survey work we are going to assess the output of the survey work (Angular error of the closure and linear error of closure). We will compare the expected angular error of closure and the linear error of closure of traverse versus actual angular and linear error of closure to check if our assumption (assumed $\pm\sim$ and $\pm d$) was correct.
- Classify the order of the checkpoints (locally and within the network) according to the *(FGCC*), 1984 standards.
- Compare the transformed coordinates of the existing map versus the checkpoints coordinates.
- Use the American Society of Civil Engineers *ASCE*, the Federal Geographic Data Committee *FGDC*, and National Map Accuracy Standard *NMAP* standards to designate the accuracy of the map.
- We will use different tools like, Arc view (extensions), Soft disk8, Autocad2000, and MathCAD.
- In our project we assume that:
  1. $\pm_r$ (Observer personal reading error) = 2.5".
  2. $\pm_p$ (Observer pointing targets error) = 2.5".
  3. $\pm_t$ (Observer centring targets error) = 0.00305m.
  4. $\pm_i$ (Observer centering instrument error) = 0.0015m.

1.6 PROJECT OUTLINE

The project consists of six chapters. The following few pages summarize the contents of each chapter.

1. Introduction

This chapter covers the general idea of the project, its importance, objectives, study case, methodology, phases or stages of the project work, outline, and problems that we find during project work…etc.

2. Maps and databases

This chapter covers definitions, purpose, care, categories (scale and types), map construction, data collection, digital maps, and digital terrain model.

3. Data collection and Map accuracy

This chapter covers introduction, primary method of data collection (mistakes, systematic, and random error), observations and errors, types of errors in data collection, secondary method of data collection, map accuracy, map testing, E.D.M (correction of E.D.M, tapes), distance reduction of control points at MSL.

4. Determination of horizontal location of points

This chapter includes control survey and classes of control survey, important , Traverses, intersection, Resection, Triangulation, Trilateration, and tingulateration, error in horizontal control points, error in vertical control points, purpose of the traverse, choice of traverse stations, type of traverse (open and close), traverse in 3-

D, error propagation in angle and distance measurement, traverse adjusted by least square, ellipses error.

5. **Field work and computation**

This chapter includes all office and surveying operations and data preparation, like GIS operations, maps evaluation and testing, and other operations as, coordinates computations from field measurements using different computer programs, like ArcView GIS, AutoCAD2002, Softdisk8, …etc.

6. **Conclusions and recommendations**

This chapter contains the conclusion of the project and recommendations.

1.7 PROJECT TIME TABLE "SCHEDULE"

Table 1.1: Project's Schedule for course (1)

	WEEK	TIME FRAME IN WEEKS													
STAGES	**NO**	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Choosing the project	2														
Preparing for the initial chapters	3														
Out line	1														
Preparing for the	6														

remaining chapters															
First preparation of project introduction report	1														
Project introduction report revision	1														
Final project introduction report printing and handing	1														

Table 1.2: Project Schedule for course (2)

Stages	**Week No.**	**Time frame in weeks**													
		1	2	3	4	5	6	7	8	9	10	11	12	13	14
Initial land survey	4														
Office calculations	4														
Preparing theory chapter	1														
Preparing field	1														

survey chapter			
Preparing calculation and analysis chapter	1		
Conclusions and recommendations	1		
Final revision and printing all chapters	1		
Handing the final report of the project	1		

1.8 PROBLEMS

To complete any project, it always passed through many obstacles. The main obstacles that we faced can be summarized as follows:

- **The instrument**

In our project it is very necessary to use the total station to make the field surveying, *total station in PPU* is very old and need calibration, to do this the uses manual of EDM is needed which is not available, and the reflector is not stable.

- **Weather conditions:**

This caused postpone of the fieldwork several time.

- **Political conditions:**

  This problem caused discontinuous work in the project.

Chapter Two

2

MAPS AND DATABASES

CHAPTER 2

MAPS AND DATABASES

2.1 DEFINITION

A map is a graphic representation of a portion of the earth's surface drawn to scale. It uses colours, symbols, and labels to represent features found on the ground. The ideal representation would be realized if every feature of the area being mapped could be shown in true shape. Obviously this is impossible, and an attempt to plot each feature true to scale would result in a product impossible to read even with the aid of a magnifying glass.

2.2 PURPOSE OF THE MAP

A map provides information on existence, location of, and the distance between the ground features, such as populated places and routes of travel and communication. It also indicates variations in terrain, heights of natural features, and the extent of vegetation cover.

2.3 CLASSIFICATIONS OF MAPS BY SCALE

Map scale is the ratio of the map distance to the corresponding true length on the ground. Map scales are generally classified as large, medium, and small. Their respective scale ranges are as follows: [Source: Reference No.2, page17]

Large scale	1:20000 and larger
Medium scale	1:20000 – 1:50000
Small scales	1:50000 and smaller

2.4 CLASSIFICATIONS OF MAPS BY TYPE

Maps are generally classified as type of the maps. Their respective maps are as follows:

- **Planimetric maps:** which graphically represent in plan such natural and artificial features as streams, lakes, boundaries, condition and culture of the land, and public and private works.

- **Topographic maps:** which include not only some or all of the preceding features but also represent the relief or contour of the ground. Maps of large areas, such as a state or country, which show the locations of cities, towns, streams, lakes, and the boundary lines of principal civil divisions, are called geographic maps. Maps of this character, which show the general location of some kind of the words of human beings, are designated by the name of the works represented. Maps of this type, which emphasize a single topic and where the entire map is devoted to showing the geographic distribution or concentration of a specific subject, are called *thematic maps*.

A topographic map shows, through the use of suitable symbols. The spatial characteristics of the earth's surface, with such natural features as hills and valleys, vegetation and rivers.

Constructed features such as buildings, roads, canals, and cultivation. The distinguishing characteristics of a topographic map, as compared with other maps, are the representation of the terrain relief.

Topographic maps are used in a variety of ways. They are necessary in the design of any engineering project that requires the consideration of elevations for gradients. They also are used for delineating the extent of a flood plain, planning for economic development, and managing natural resources.

The preparation of general topographic maps traditionally has been a function of governmental agencies. However, the rapid development of computer-based tools for terrain modelling and the increasing availability of high- quality field data enable nearly any user of topographic information to create detailed, specialized topographic maps using a desktop computer.

Topographic maps show shorelines, locations and depths of soundings or lines of equal depth, bottom conditions, and sufficient planimetric or topographic features of lands adjacent to the shores to interrelate the positions of the surface with the underwater features. Charts of the U.S. National ocean survey are good examples of hydrographic maps. Such maps form an important part of the basic information required for the production of environmental impact statements.

3. *Photomap*. This is a reproduction of an aerial photograph upon which grid lines, marginal data, place names, route numbers, important elevations, boundaries, and approximate scale and directions have been added.
4. *Photomosaic*. This is an assembly of aerial photographs that is commonly called a mosaic in topographic usage. Mosaics are useful when time does not permit the compilation of a more accurate map. The accuracy of a mosaic depends on the method employed in its preparation and may vary from simply a good pictorial effect of the ground to that of a planimetric map.
5. *Terrain Model*. This is a scale model of the terrain showing features, and in large-scale models showing industrial and cultural shapes. It provides a means for visualizing the terrain for planning or indoctrination purposes and for briefing on assault landings.
6. *Thematic Maps*. These are maps for special purposes, such as trafficability, communications, and assault maps. They are usually in the form of an overprint in the scales smaller than 1:100,000 but larger than 1:1,000,000. A special purpose map is one that has been designed or modified to give information not covered on a standard map.

2.5 MAP CONSTRUCTION

Maps are constructed using data acquired in the field by yield survey methods, such as radial surveys with a total station system or kinematics GPS surveys. Information from these surveys, stored in electronic field data collectors supplemented by notes in conventional field notebooks, is electronically transferred or keyed into an electronic computer in the office for processing and plotting using CAD. Maps more frequently are carried by either aircraft or satellite, note that, for a map compiled by photogrammetric a carried methods, a sufficient number of ground control points of known positions must be identifiable in the photographic or sensed record to allow scaling the map and selecting the proper datum.

2.6 DATA COLLECTION

Data collection plays a very important role in any GIS or computer mapping system. Data for GIS are obtained in many ways, like existing datasets, map digitizing, and direct data collection by field surveying, figure (2.1).

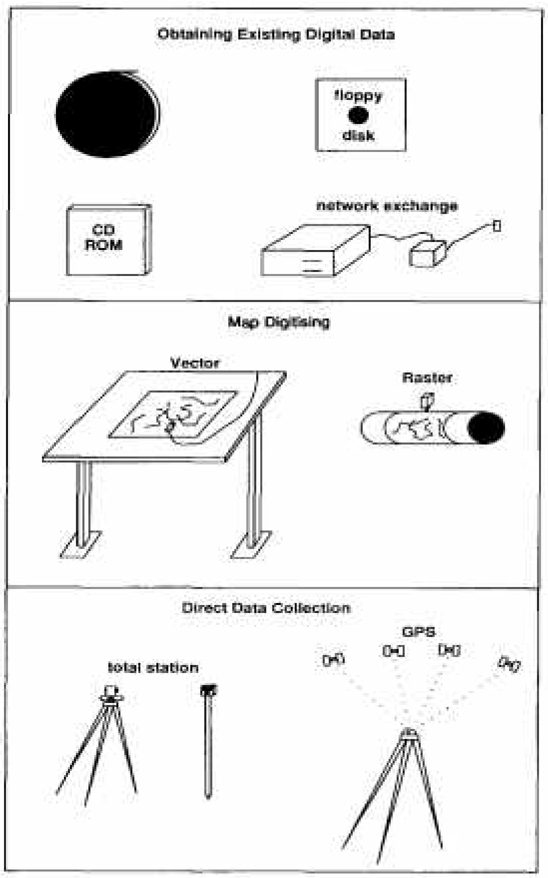


Figure (2.1): The major data collection methods employed in GIS.

2.6.1 Existing datasets:

Many GIS databases consist of sets of information called *layers*. Each layer represents a particular type of geographic data. For example, one layer may include information on the streets in an area. Another layer may contain information on the soil in that area, while another records elevation. The GIS can combine these layers into one image, showing how the streets, soil, and elevation relate to one another.

2.6.2 Map digitizing:

One of the important steps in creating digital maps is conversion of traditional paper maps (or analogy maps) into digital GIS data layers. Not only do the data have to be transferred into a digital form, but they also have to be vectorized so that GIS software would be able to distinguish between individual elements such as lines, points, and polygons. There are several ways to do this:

- Manual digitizing from analogy maps.
- Scanning with further "heads-up" digitizing or automatic vectorization.

2.6.3 Direct data collection:

Data collected in the field or interpreted from aerial photography or other remote sensor imagery can be directly entered into a GIS or computer mapping database. Many types of direct data collection are now used, but direct collection of survey and photogrammetric data have had the greatest impact on cartography and GIS.

2.6.4 Surveying data collection:

Modern surveying instruments known as total stations simultaneously measure horizontal and vertical angles, as well as the distance between the instrument and a second point, figure (2.2). When the instrument is placed over a triangulation point or some other position with known geographic coordinates and elevation, coordinates and elevation of the second and additional points can be computed directly by the instrument and stored in a digital form that can later be read into a computer mapping program or GIS.

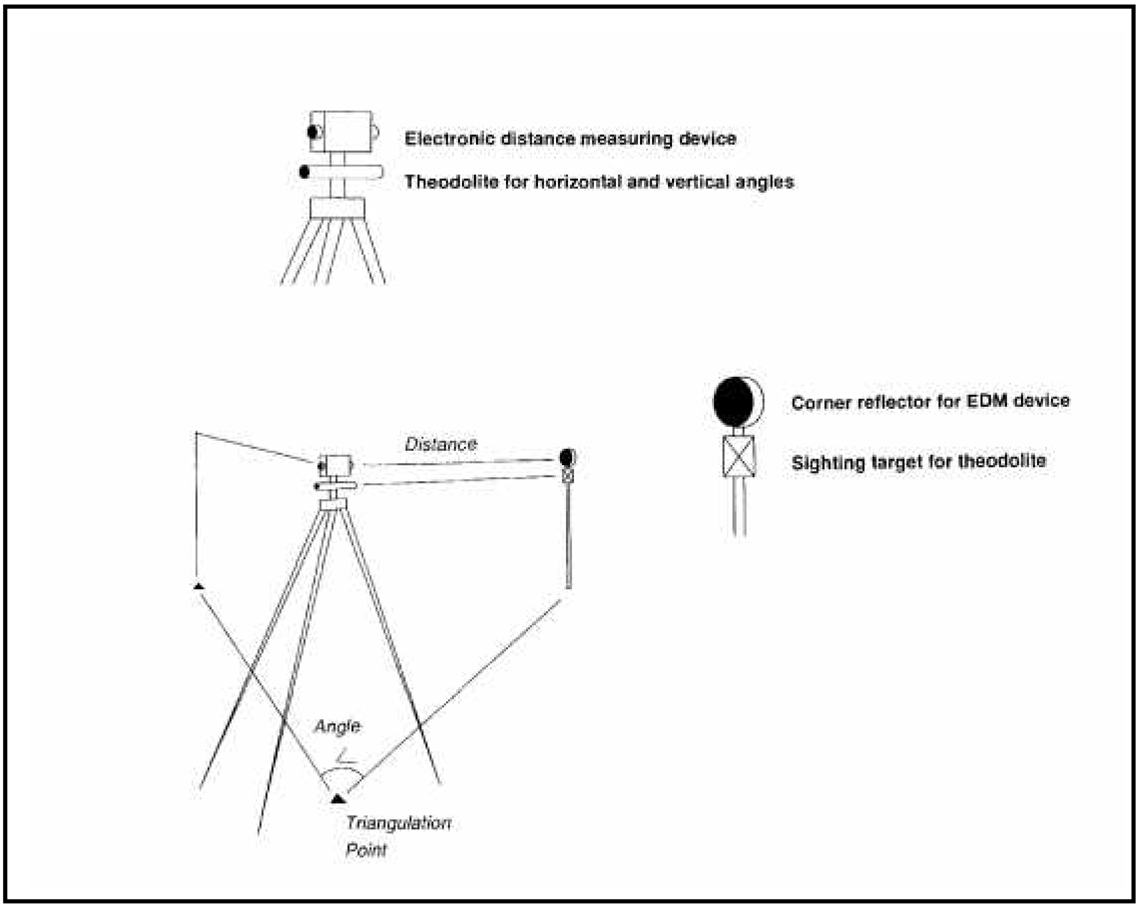


Figure (2.2): Key components of the total station-surveying instrument

1. Global Navigation Systems:

Similar direct collection and digital storage of horizontal and vertical positions is possible using the US Global Positioning Satellite (GPS) system, the Global Navigation Satellite System (GLONASS), or the two in combination. In simple terms, these global navigation systems are based on twenty or more identical navigation satellites placed in high altitude (17000 km) orbits arranged so that signals sent from four or more satellites can be simultaneously received anywhere on earth, figure (2.3).

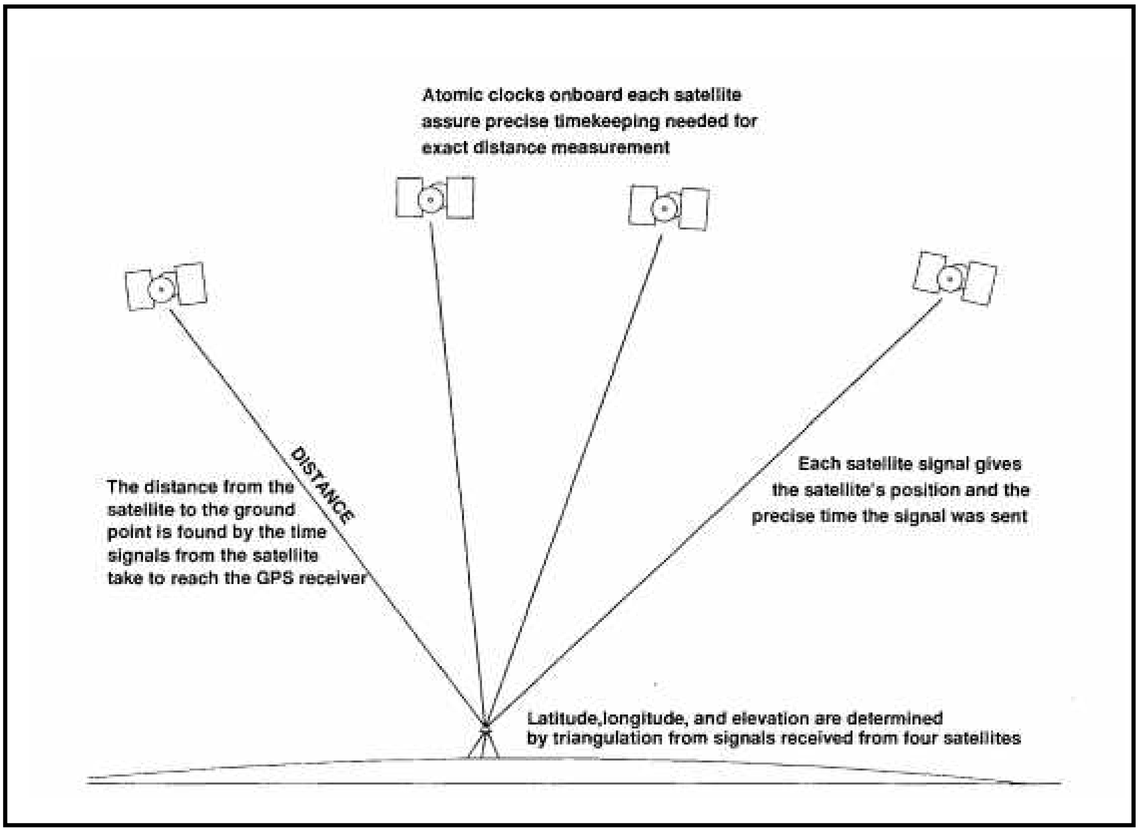


Figure (2.3): Signals received from four or more global positioning satellites (GPS) allow latitude, longitude, and elevation to be precisely determined anywhere on earth

2. Photogrammetric data collection:

Locations of roads, coastlines, rivers, lakes, houses, contour lines and similar data obtained by photogrammetric measures from aerial photographs are routinely entered directly into cartographic and GIS databases. Modern stereoplotters figure (2-4), allows operators to trace the locations of these features into a cartographic data collection program in a manner similar to using a vector-digitising tablet.

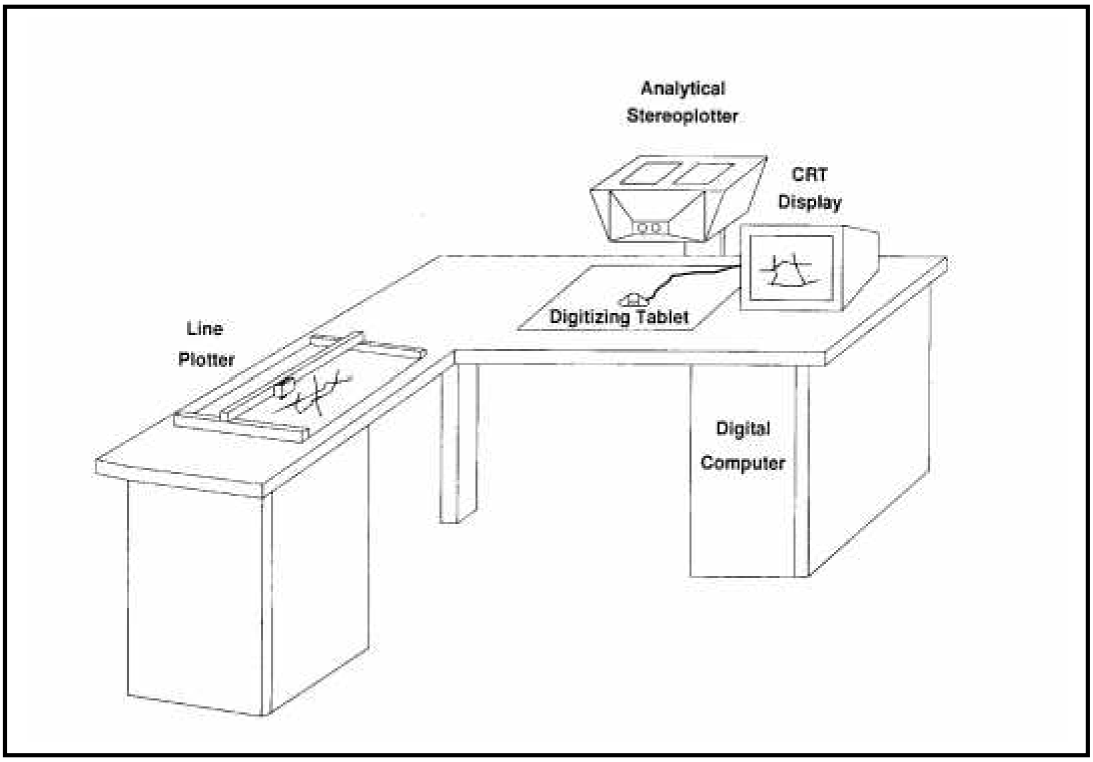


Figure (2.4): Components of modern photogrammetric stereoplotting instrument

3. Remote Sensing "Satellite Images":

Remotely sensed imagery from aircraft and satellites represent one of the fastest-growing sources for raster GIS data. Remote sensing is the study of phenomena sensed from air or satellites. The use of remote sensing provides scientists with an opportunity to interpret data as seen from space. Remote sensing is therefore widely used in such locations as the arctic zones, deserts, forested areas, marshlands and mountainous regions, where small-scale maps of an adequate quality can be reproduced relatively from satellite images. In general, remote sensing is suitable for studying extensive areas with difficult or inhospitable terrain.

Satellite and aerial remote sensing provide an almost infinite pool of information. They provide the experienced interpreter with knowledge of ground conditions, including geomorphology, vegetation, soils, and mineral composition. A great deal of time and cost can be saved, through remote sensing, by reducing the necessary amount of fieldwork. Taken at regular intervals and over a long period of time, satellite remote sensing can also help scientists to identify the extent of change in ground conditions over the seasons, such as seasonal soil moisture changes. This is particularly useful in mapping areas that witness frequent changes in land cover, like agricultural lands, tidal areas, and marshlands.

2.7 DIGITAL MAPS

Cartographic objects are the basic geometrical components of cartographic and GIS databases that define three fundamental geometrical entities: points, lines, and homogeneous areas "polygons" that we use to represent environmental features. Figure (2.5) includes the following:

2.7.1 Vector objects:

• *Point*: a zero-dimensional object specifying geometric location by a set of coordinates.

• *Node*: a zero-dimensional object serving as a topological (line) junction or endpoint.

• *Line segment*: a straight line between two points.

• *String*: a sequence of line segments without nodes.

• *Chain*: a sequence of line segments with beginning and ending nodes.

• *Ring*: a sequence of chains or strings that form a closed loop.

• *Polygon*: a ring and its interior area.

2.7.2 Raster objects:

• *Pixel*: a two-dimensional picture element that is the smallest non-divisible element in an image.

• *Grid cell*: a two-dimensional object representing an element of a regular surface tessellation.

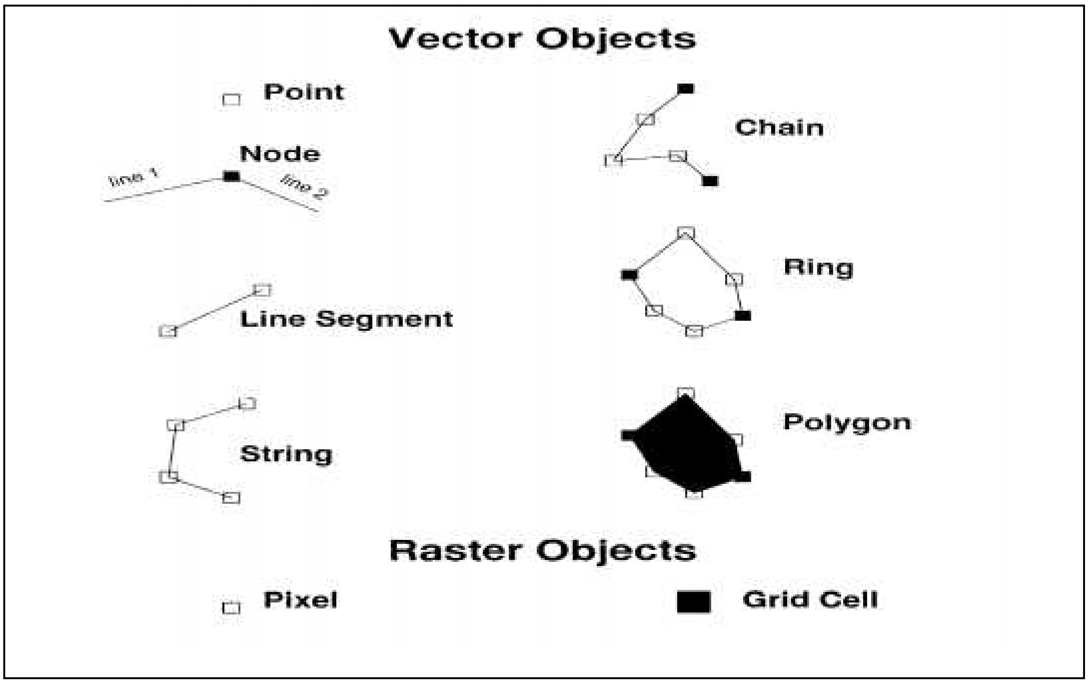


Figure (2.5): Cartographic objects widely used in GIS

2.8 CARTOGRAPHIC DATA STRUCTURES

Cartographic objects describing data themes (also known as data layers or coverage's) must be structured within the GIS database so that data analyses utilizing data from one or more themes can be performed accurately and efficiently.

Both the location and attribute data describing each object must be treated, and the locational data must be structured so as to retain the topological relationships of adjacency and connectivity that inherently exist among the features comprising each theme.

2.8.1 Raster data structures:

A surface tessellation is a sub division of a two- or three-dimensional surface into a set of basic geometrical figures that completely cover the surface without gaps or overlaps. Only three regular geometrical forms - square, equilateral triangles and hexagons - will give a two-dimensional surface tessellation, figure (2.6).

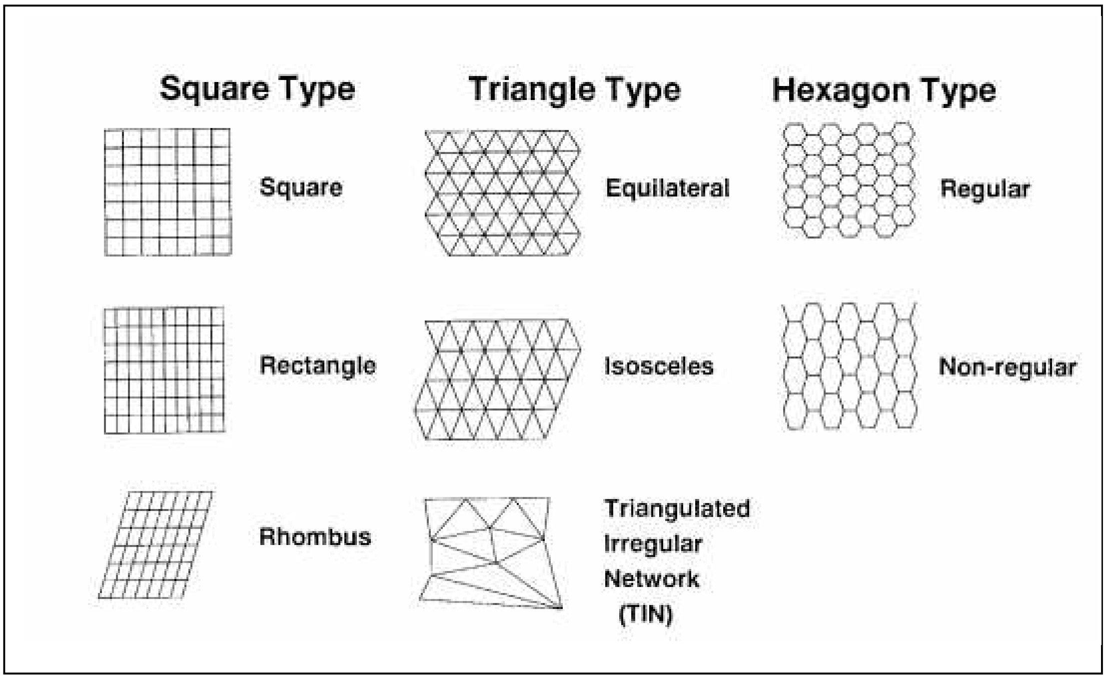


Figure (2.6): Two-dimensional surface tessellations

Square tessellations are used almost exclusively in computer mapping and GIS, the major exception being the subdivision of the true three-dimensional earth surface into a triangular irregular network (TIN structure). The square structure is used most often for several reasons:

Two-dimensional arrays holding GIS data and electronic display screens are structured in this way.

The column-row locations of two pixels inherently show if they are adjacent and connected. Squares can be subdivided into 2 x 2, 3 x 3 … arrays of smaller squares, which can be further subdivided.

2.8.2 Vector data structures:

For the simple drawing of maps at a given scale on a certain map projection using a computer controlled plotter, data need only be structured as sets of points, strings, and rings, figure (2.7).

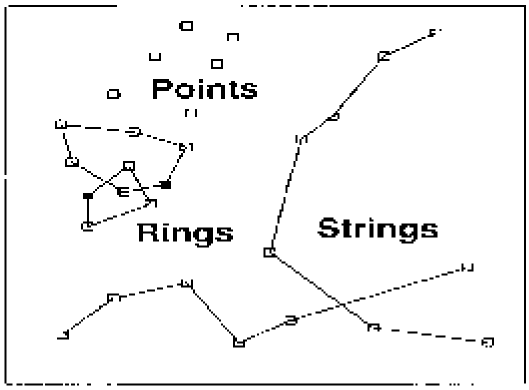
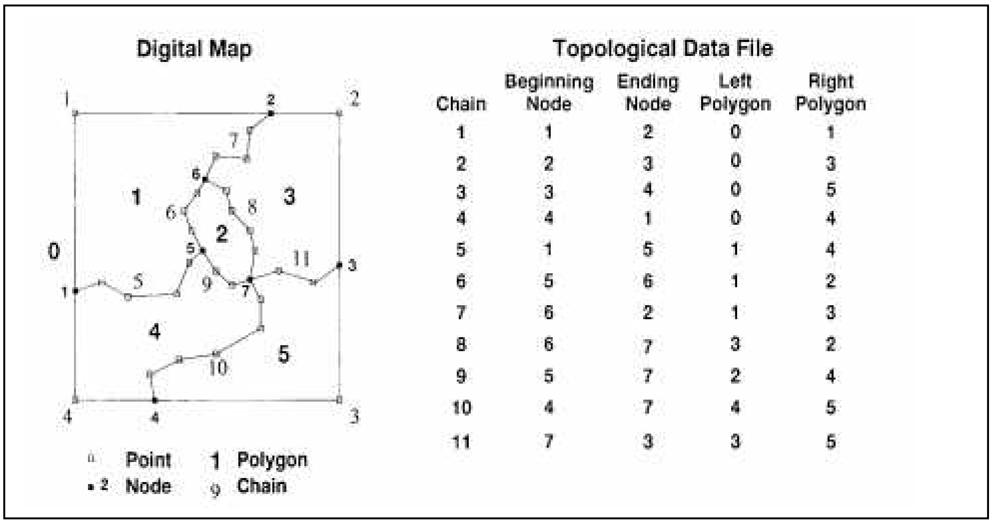


Figure (2.7): Entity data structure for simple computer mapping.

This is often called an entity structure, because each point, line, and area boundary is separately encoded without regard to neighboring features. One disadvantage of the entity structure is that all interior area boundaries must occur twice in the database and be digitized identically in order to avoid tiny gaps and overlap slivers along boundary lines. This data redundancy, and the fact that GIS analyses such as map overlay operations require topological information, makes the entity structure less than ideal for GIS.

The topological vector data structure is based on encoding all lines as chains, and adding information as to the polygons to the left and right side of the chain when viewed from the beginning to ending node, as in figure (2.8).



Chain	Beginning Node	Ending Node	Left Polygon	Right Polygon
1	1	2	0	1
2	2	3	0	3
3	3	4	0	5
4	4	1	0	4
5	1	5	1	4
6	5	6	1	2
7	6	2	1	3
8	6	7	3	2
9	5	7	2	4
10	4	7	4	5
11	7	3	3	5

Figure (2.8): Topological data structure for lines and areas based on chains and chain topology

This adjacency and connectivity information allows polygons to be built from the chain data, and also for automatic error checking of line data. Coupled with the elimination of redundant line data and the ability to carry out complex GIS analyses, the topological vector data structure makes modern GIS possible.

2.9 DIGITAL TERRAIN MODELS

DTMs generally are organized such that the mass points lie in a regular grid pattern or they represent vertices of local triangular paths in an array referred to as a triangulated irregular network. [Source: Reference No.2]

The DEM is relatively recent type of product, in which a very dense network of point of known X, Y, and Z coordinate represents the terrain surface numerically. This product has been given several names: digital terrain model (DTM), digital elevation model (DEM), digital height model (DHM), or digital terrain elevation data (DTED). The DTM may be product in a photogrammetric analytical plotter, where the data are collected in profile form with equal spatial interval or time interval during digitizing, or selectively at terrain break points as designated. The data also may be obtained as Z values for an X, Y grid with equal X and Y spacing. Another possibility is to digitize along couture lines, where a large set of X and Y values are developed for each given Z value. The EDM may be produced in a digital potogrammetric workstation as will. In fact, it is the first digital product from a workstation in which substantial automation has been implemented. A hierarchical procedure of image matching often is used to produce automatically the elevation as designated grid points. This, however, usually is following by manual editing.

Recently, the U.S.G.S also provided the same data contained in the printed map series in digital form as Digital Line Graph (DLG) files, although the printed maps are regularly photo revised, it is common for the latest quadrangle maps not to have been revised in several years. For engineering applications in rapidly developing areas, this time lag may not be acceptable.

The U.S.G.S. currently is in the process of developing a new, large-scale digital orthophoto quadrangle (DOQ) and quarter-quad (DOQQ) map series for the entire country. These new orthophoto maps will have an l-m ground sample (pixel) resolution and will be provided in a UTM system based on NAD 83. The new map series combines the information content of a photograph and the geometric qualities of a standard map and will be uniquely suited for use in spatial information systems.

Chapter Three

3

DATA COLLECTION AND MAP ACCURACY

CHAPTER 3

DATA COLLECTION ERRORS AND MAP ACCURACY

3.1 INTRODUCTION

Measurements are essential to the functions of the surveyor. The surveyor's task is to design the survey; plan its field operations; designate the amount, type, and acquisition techniques of the measurements; and then adjusts and analyzes these measurements to arrive at the required survey results. It is important, then, that the individual studying surveying understand the basic idea of a measurement or an observation (here, the two terms are used interchangeably).

Except for counting, measuring entails a physical operation that usually consists of several more elementary ones, such as preparation (instrument setup, calibration, or both), pointing, matching, and comparing. Yet, the result of these physical operations is assigned a numerical value and called the measurement. Therefore, it is important to note that a measurement really is an indirect thing, even though in some simple instances it may appear to be direct. Consider, for example, the simple task of determining the length of a line using a measuring tape, this operation involves several steps: setting up the tape and stretching it, aligning the zero mark to the left end of the line, and observing the reading on the tape at the right end. The value of the distance as obtained by subtracting 0 from the second reading is what we call the measurement, although actually two alignments have been made, to make this point clear, visualize the tape simply aligned next to the line and the tape read opposite to its end. In this case, the reading on the left end may be 11.9 and the one on the right end 52.2, with the net length measurement of 40.2

3.2 TYPES OF ERROR

An error is a difference between the true value and the observed value of a quantity caused by the imperfection of equipment, by environmental effects, or due to the imperfection in the senses of the observer.

Errors are generally classified into three types: [Source: Reference No.2]

1. Mistakes or blunders.
2. Systematic errors.
3. Random errors.

3.2.1 Mistakes

- Are caused by carelessness or inattention of the observer in using equipment, reading scales or dials or in recording the observations. For example, the observer may bisect the wrong target in angle observations. Or may record observations by transposing numbers, e.g. by writing 65.25 instead of 56.25. They could also be introduced by misidentification of a control point in an aerial photograph. Gross errors may also be caused by failure of equipment. Observation fraught with gross errors is useless. Therefore, every attempt must be made to eliminate gross errors. Normally, observation procedures are designed in such a way that one can detect gross errors during or immediately after the observations are taken.
- Some of the techniques used in ducting and eliminating gross errors include taking multiple reading on scales and checking for consistency using simple

geometric and algebraic checks, repeating the whole measurement and checking for consistency, etc.

- In a statistic sense, gross errors are observations, which cannot be considered to belong to the same sample as the rest of the observations. Therefore, the elimination of gross errors or blunders or mistakes is vitally important.

3.2.2 Systematic errors

- Occur in accordance with some deterministic system, which, if known, may be represented by some functional relationship. For example, observed slope distances if not reduced to the ellipsoid will introduce systematic errors.
- There is functional relationship between the observed distance, geoids ellipsoid separation, and the heights of the points between which the distances observed. In surveying, geodesy, and photogrammetry systematic errors occur because of environmental effects, instrumental imperfections, and human limitations.
- Some of the environmental effects are humidity, temperature, and pressure changes. These factors affect distance measurements, angle measurements, and GPS satellite observation, among others. Instrumental effects include lack of proper calibration and adjustment of the instrument as well as imperfection in the construction of the instrument, e.g., nonuniform graduations of the linear and circular scales. Systematic errors must be detected and observations must be corrected for systematic errors or they must be modelled by mathematical model.
- In a statistical sense, systematic errors introduce bias in the Observations. Unlike gross errors, they cannot be detected or eliminated by repeated observations. Therefore, if systematic errors are present, the measurements may be precise but they will not be accurate.

3.2.3 Random errors

- After all the gross errors and systematic errors are removed there will still remain some variants in the observations. These remaining variations in observations (which are small in magnitude) are called random errors. They cannot be represented by a functional relationship based on a deterministic model. Random errors occur due to the imperfection of the instrument and observed quantity will be either too small or too large every time it is observed.

Random errors possess the following characteristics:

- Positive and negative errors occur with the same frequency.
- Small errors occur more often than large errors.
- Large errors rarely occur.

Random errors are treated systematically by using a stochastically model. After mistakes are detected and eliminate and all sources of systematic errors are identified and corrected, the values of the observations will be free of any bias and regarded as sample values for random variables.

A random variable may be defined as a variable that takes on several possible values and a probability is associated with each value.

Probability may be defined as the number of chances for success divided by the total number of chances or as the limit value to which the relative frequency of occurrence tends as the number of repetitions is increased indefinitely. As a sample illustration, consider the experiment of throwing a die and noting the number of dots on the top face. That number is a random variable, because there are six possible values, from 1 to 6. For example, the probability that three dots will occur tends to be 1/6 as the number of throws gets to be very large. This probability value of 1/6 or 0.166 is the limit of the relative frequency. Or the ratio between the numbers of times three dots show up and the total number of throws.

Survey measurement, ssuch as a distance or angle, after mistakes are eliminated and systematic errors corrected, is a random variable such as the number of dots in the die example. If the nominal value of an angle is 41°13'36" and the angle is measured 20 times, it is not unusual to get values for each of the measurements that differ slightly from the nominal angle. Each of these values has a probability that it will occur. The closer the value approaches $41^o13'36''$ the higher is the probability; and the farther away it is, the lower is the probability.

In the past, the value $41^o13'36''$ was designated the “ true value,” which was never known. Then, when an observation was given that, owing to random variability, was different from the true value, an error was defined as:

Error = measured value – true value

And a correction, which is the negative of the error, was defined as:

Correction = true value – measured value

Whereas for systematic effects, the concept of error and correction is reasonable, in the case of random variation it is not. Since there is no reason to say that errors have been committed. The so–called random error actually is nothing but a random variable, because it represents the difference between a random variable, the measurement, and a constant, the true value. The ideal value of the error is 0 (which in statistics is called the expected value): that for the observation is the true value (in statistics called the expectation or distribution mean « , as will be explained later. The variation of the random errors around – is identical to the variation of the observations around the expectation « or the true value. Thus, it is better to talk about the observations themselves and seek better estimates for these observations than discuss errors, because, strictly speaking, the values being analyzed are not error. Than discuss errors, because, strictly speaking, the values being analyzed are not errors.

Another classically used term is discrepancy, which is the difference between two measurements of the same variable. Best value, most probable value, and corrected value are all terms that refer to a new estimate of a random variable in the presence of redundancy. Or have more measurements than the minimum necessary. Such and estimate usually is obtained by some adjustment technique, such as least squares, if the random variable is x, the estimate from the adjustment is called the least–squares estimate or sometimes the adjusted value, denoted by x. the residual has been defined by equation and will be used in the same sense as it has been in the past. The deviation is simply the negative of the residual and may be used on occasion in the course of statistical computation.

3.3 TYPES OF ERRORS IN DATA COLLECTION

There are two classifications of errors in data collection:

- Errors in primary methods of data collection.
- Errors in Secondary methods of data collections.

3.3.1 Errors in primary methods of data collection

The primary methods of data collection include the techniques of geodesy, photo grammetry, and surveying. All these techniques include random, systematic, and gross errors. However, the methods of observations as well as computation are designed in such a way that gross errors are eliminated, systematic errors are either corrected or are mathematically modelled, and random errors are treated systematically by stochastic models, for example, by using the method of least squares. [Source: Reference No.2]

Errors introduced in the primary method of data collection include the following types:

- Personal errors.
- Instrumental errors.
- Environmental errors.

3.3.1.1 Personal errors

Occur because no observer (surveyor or geodist its or photogrammetist) has perfect senses if sight and touch.

This type of error includes reading errors, centering errors .as well as bisection errors. This also includes the personal equation, which involves how a particular individual estimates the readings between graduations. Personal errors could be blunders and systematic as well as random errors.

3.3.1.2 Instrumental errors

Include errors caused by imperfect instrument construction, or lack of adequate instrument adjustment or calibration prior to its use in data collection. Error introduced by instrumental effects is mainly systematic in nature.

3.3.1.3 Environmental errors

Are primarily caused by variations temperature, pressure, humidity, magnetic variations, and obstruction of signals, winds, and illumination at the time of observation.

3.3.2 Errors in Secondary methods of data collections

The secondary methods of data collection include all the errors contained in the primary methods. In addition the secondary methods include the following errors:

3.3.2.1 Errors in plotting control

The first step involved in making a map is to plot the control points. The root mean square error (RMSE) involved in this process, (e_1), varies between 0.17 to 0.32 mm for coordinate graphs attached to photogrammetry plotters. The error introduced during a control survey using for example the global positioning system (GPS) can be ignored at plotting scale because one achieves centimeter level accuracy.

3.3.2.2 Compilation error

Compilation of topographical maps entails bringing data from various sources to a common scale by photography the error introduced in this process (malign 1989) is (e_2 = k e) the value of (e_2) ranges between 0:30 mm and 0:32 mm where (e) is the error in the detail survey which ranges from 5 m to 7.5 m if the detail is collected by photogrammetry method for a map with a scale 1:25,000, and (k) is the amount of reduction in scale from that the plotted detail to that of the compiling manuscript .it should also be noted that in digital mapping point features will have a different accuracy than line features. Normally, well defined point features can be compiled more accurately than line features moreover even within line features the accuracy of compilation will vary depending on the definition and thickness the features.

3.3.2.3 Errors introduced in drawing

The drawing error, (e_3), is usually introduced at fair drawing stage and is quoted as ranging from 0.06 mm and 0.18 mm.

3.3.2.4 Errors dut to generalization

The generalization error is very difficult to quantify because the amount of error introduced depends on the type of features and also on the character (or complexity) of the feature. The error could range from substantial for some features.

Error due to deliberate features to be portrayed on a map is so close that they cannot be plotted in their proper position without overlapping. Therefore, they are displaced at the time of plotting to make the map legible .for example if there is a road one side of a river and railway line on the other side, then three features cannot be plotted without displacing some them. The smaller scale of a map the larger the displacement. Again this error could be substantial depending on map scale and the proximity of the features to be portrayed.

3.3.2.5 Errors in map reproduction

The RMSE in map reproduction, (e_5), varies between 0.1 mm and 0.2 mm.

3.3.2.6 Error in color registration

A colour map is reproduced from a series of metal printing plates, which are used to print on a paper one colour at a time. The RMS error introduced in proper registration (e_7) varies between 0.17 and 0.30 mm.

3.3.2.7 Errors introduced by the deformation

Maps are normally printed on paper the dimensions of paper change with changes in humidity and temperature when an increase in humidity, the moisture content of paper may increase from 0 percent to 25 percent with a corresponding change in paper dimensions of as much as 1.6 percent at room temperature. The paper will not return to its original size even if the humidity is reduced because the rates of expansion and shrinkage are not the same. A 36-inch long paper map can change by as much as 0.576 inches due to change humidity.

Nearly all materials increase in dimension when heated a decrease when cooled paper is no exception to this rule at time of printing the paper temperature is high therefore can be stretched up to 1.5 percent in length and 2.5 width after the paper dries and cools it shrinkage by 0.5 percent in length and 0.75 percent in width. The net change in the dimensions of the paper map after printing and cooling may be 1.25 percent in length and 2.5 percent in width.

3.3.2.8 Error introduced by the use of uniform scale

The scale quoted in a map is what is known as the principal scale, which is true. For example, for the Lambert conformal projection the principal scale is true only along standard parallels the scale is too small between the parallels and too large outside the parallels therefore one should use the proper scale factor correction when digitizing a map or when measuring distances from maps.

When information from different maps is collected then one has to make sure that they are using the same map projections and are of compatible scales. Revised “old” maps may have used different map projections in a new edition but this may not have been stated in the peripheral information.

3.3.2.9 Error introduced due to exaggeration

In order to increase the communicative value and legibility of a map, features are sometimes exaggerated because they can not be portrayed at their proper dimensions for example a boundary line normally does not have a width yet when it is plotted on a map it occupies a substantial width. Some features are more exaggerated than others depending on the purpose of a map. For example roads are, exaggerated on a road map. Error due to feature exaggeration could be substantial depending on the scale and purpose of the map and the type of feature involved. Again, one must point out that not all features will have this kind of error.

3.3.2.10 error in digitization

Digitization and scanning errors depend on the following factors:

- Width of the feature.
- Skill of the operator.
- Complexity of the feature.
- Resolution of the digitizer.
- Density of the features.

When digitizing a thick line it is difficult to continually place the cursor on the middle of the line .the operator is also likely to make more errors when digitizing in areas where the features are dense for example contour lines in mountainous areas. The operator is also likely to make errors when he/she is tired. Note that errors for point features will not be the same as for linear features the digitization error is quoted as e_{10} = 0.25 mm. Line following techniques and scanners perhaps introduce fewer planimetric errors but errors in feature tagging could be higher in the case of scanning.

3.3.11 Total error in secondary methods errors

It is very difficult if not impossible to assess the total error introduced in the secondary methods of data collection because we do not know the functional relationship among the various error introduced at different stages of the mapping processes dimensional instability of the medium, and digitization as summing that a linear relationship exists between the total error and the individual errors, the total error may be computed by using the law of propagation of errors, the value of the error in each part in secondary methods errors.

Total error = $(e_1^2 + e_2^2 + e_3^2 + e_5^2 + e_6^2 + e_7^2 + e_{10}^2)^{1/2}$

Worst-case scenario

Total (RMS) error = $(0.32^2 + 0.32^2 + 0.18^2 + 0.2^2 + 0.30^2 + 0.48^2 + 0.25^2)^{1/2}$

=0.81 mm at map scale.

Best-case scenario

Total (RMS) error = $(0.01^2 + 0.30^2 + 0.06^2 + 0.10^2 + 0.17^2 + 0.24^2 + 0.25^2)^{1/2}$

= 0.50 mm at map scale.

The above computations show that the positions could be off by several meters if we use maps at scales of 1:24,000. This obviously is an unacceptable error for many applications.

3.4 MEASURES OF QUALITY

Several terms are used to describe the quality of measurements and the quantities derived from them. The following are those most commonly used in surveying:

Accuracy*:* the term accuracy refers to the closeness between measurements and there true values, the further a measurement is from its true value, the less accurate it is.

Precision: As opposed to accuracy, the term precision pertains to the closeness to one another of a set of repeated observations of a random variable, if such observations are closely clustered together, then they said to have been obtained with high precision. It should be apparent, then, that observations might be precise but not accurate, if they are closely grouped together but about a value that is different from the true value by a significant amount. Also, observations may be accurate but not precise if they are well distributed about the true value but dispersed significantly from one another. Finally, observations will be both precise and accurate if they are closely grouped around the expected value (or the distribution mean).

An example often used to demonstrate the difference between the two concepts of accuracy and precision is the grouping of rifle shots on a target.

Different groupings that is possible to obtain. From the preceding discussion, group (a) is both accurate and precise, group (b) is precise but not accurate, and group (c) is accurate but not precise, a hard notion to accept is that case (c) in fact is accurate, even though the scatter between the different shots is rather large. A justification that may help is that we can visualize that the center of mass (which is the true value). The most commonly used measure is the variance or its positive square root, the standard deviation.

Weight: It is easy to see that the higher is the precision, the smaller is the variance. To avoid the apparent reversal in meaning, the term weight is used to express a quantity that is proportional to the reciprocal of the variance (for uncorrelated or independent random variables). It is denoted by w.

Relative precision: The term relative precision refers to the ratio of the measure of precision, usually the standard deviation, to the quantity measured or estimated. For example, if a distance s is measured with a standard deviation $\pm s$.the relative precision is$\pm s / s$.

Ratio of misclosure: in traverse computations the ratio of misclosur (ROM) is given by

ROM = dc/S

Where d_c is the misclosure in m or ft and S is the length of traverse in m or ft. Although some practicing surveyors to classify the quality of traverses perfumed by traditional methods have used the ratio of misclosure. It has no statistical basis and its use therefore is not recommended.

Relative line accuracy: When the propagated standard deviation, $\pm v$, is given for a line of length S_{ij}, between points i and j. Then the relative line accuracy (RLA) is given by

$$RLA = \frac{\pm_{ij}}{s_{ij}}$$

Note that the RLA is just a special example of the general concept of relative precision.

Mean square error: When the so-called true values are available to compare with calculated values. The mean square error (MSE) is given by

$$MSE = \left[\sum (x - {}^{2})^{2}\right] / n$$

In which x is the measured value, t is the true value, and n is the number of measurements.

Root mean square positional error: Horizontal positions are frequently specified by x and y rectangular coordinates. The root mean square positional error (RMSPE) is

$$RMSPE = (MSEx + MSEy)^{1/2}$$

In which x the measured value, ^{2} is the true value, and n is the number of measurements.

In which $MSEx = 1/n \sum (Xmeas - Xtrue)^{2}$

$MSEy = 1/n \sum (Ymeas – Ytrue)^{2}$

Each is compute with equation above. The RMSPE is often used in determining map accuracy.

3.5 MAP ACCURACY

3.5.1 National Map Accuracy Standards

Horizontal accuracy: For maps on publication scales larger than 1:20,000, not more than 10 percent of the points tested shall be in error by more than 1/30 inch, measured on the publication scale; for maps on publication scales of 1:20,000 or smaller, 1/50 inch

Vertical accuracy: As applied to contour maps on all publication scales, shall be such that not more than 10 percent of the elevations tested shall be in error by more than one-half the contour interval.

3.5.2 United State Geological Survey (USGS) Standards

The horizontal accuracy standard requires that the positions of 90 percent of all points tested must be accurate within 1/50 of an inch (0.05 centimeters) on the map. At 1:24,000 scales, 1/50 of an inch is 40 feet (12.2 meters).

The vertical accuracy standard requires that the elevation of 90 percent of all points tested must be correct within half of the contour interval. On a map with a contour interval of 10 feet, the map must correctly show 90 percent of all points tested within 5 feet (1.5 meters) of the actual elevation.

3.5.3 American Society for Photogrammetry and Remote Sensing (ASPRS, 1989)

Horizontal accuracy: is expressed in terms of root mean square error (RMS). *Vertical Accuracy:* is defined as the (RMS) in elevation for will defined *checkpoint, are set one-third the contour interval.*

$$MSE_x = \left[\sum (x - {}^{2})^2\right] \Big/ n \qquad MSE_y = \left[\sum (y - {}^{2})^2\right] \Big/ n$$

Table 3.1: Planimetric coordinate accuracy for well defines point.

PLANIMETRIC (X OR Y) LIMITING RMS ERROR (m)	TYPICAL MAP SCALE
0.0125	1/50
0.025	1/100
0.050	1/200
0.125	1/500
0.25	1/1000
0.50	1/2000
1.00	1/4000
1.25	1/5000
2.50	1/10000
5.00	1/20000

3.5.4 Federal Geographic Data Committee (FGDC, 1998)

The FGDC uses root-mean-square error (RMSE) to estimate positional accuracy.

- *Horizontal accuracy:*

$$RMSE_x = \sqrt{\sum (x - {}^{2})^2 / n} \qquad RMSE_y = \sqrt{\sum (y - {}^{2})^2 / n}$$

$$RMSE_T = \sqrt{MSE_x^2 + MSE_y^2}$$

Horizontal accuracy computed at the 95% confidence level

- *Vertical Accuracy:*

$$RMSE_z = \sqrt{\sum (x - {}^{2})^2 / n}$$

Vertical accuracy computed at the 95% confidence level.

3.6 MAP TESTING

Maps can be tested for accuracy by using several techniques and each agency uses its own methods.

3.6.1 American Society for Photogrammetry and Remote Sensing (ASPRS)

The ASPRS (1989) accuracy standard recommend that the conventional rectangular topographic maps be field checked with a minimum of 20 checkpoint, with 20% of these point, located in each quadrant of the map sheet, are spaced at interval of at least 10% of the sheet diagonal.

Check surveys should conduct according to the Federal Geodetic Control Subcommittee (FGCS) standard for vertical and horizontal network (FGCS 1984; 1994).

Table 3.2: Horizontal control accuracy standards for traverses (FGCS, 1994)

ORDER CLASS	FIRST	SECOND		THIRD	
		I	II	I	II
Azimuth closure at Azimuth checkpoint	$1.7\sqrt{N}$	$3\sqrt{N}$	$14.5\sqrt{N}$	$10\sqrt{N}$	$12\sqrt{N}$
Position closure after Azimuth adjustment	$0.04\sqrt{K}$ OR *1 : 100000*	$0.08\sqrt{K}$ OR *1 : 50000*	$0.2\sqrt{K}$ OR *1 : 20000*	$0.4\sqrt{K}$ OR *1 : 10000*	$0.8\sqrt{K}$ OR *1 : 5000*

Where: N is number of segment

K is root distance in Km

3.6.2 United State Geological Survey (USGS)

The USGS experts select 20 or more well defined points; using sophisticated surveying techniques to determine positions. Field survey methods are the only tests accepted for official accuracy testing.

Positions must be obtained by surveys of a higher accuracy. Vertical tests are run separately to determine precise elevations. The mapped positions are checked against the field determined positions results.

3.6.3 National Standard for Spatial Data Accuracy (NSSDA)

A minimum of 20 checkpoints shall be tested, distributed to reflect the geographic area of interest and the distribution of error in the dataset. When 20 points are tested, the 95% confidence level allows one point to fail the threshold given in product specifications.

Chapter Four

4

DETERMINATION OF HORIZONTAL LOCATION OF POINTS

CHAPTER 4

DETERMINATION OF HORIZONTAL LOCATION OF POINTS

4.1 FIELD SURVEY

Survey has to deal with the determination of the relative spatial location of points on or near the surface of the earth. It is the art of measuring slope and horizontal and vertical distances between objects, of measuring angles between lines, of determining the directions of lines, and of establishing point locations by predetermined angular and linear measurements.

With the advance of microelectronic and computing technology, the use of a total station accompanied by a data logger to capture field data becomes very popular for most land surveyors. Raw data captured from a total station are transmitted to a data logger instantaneously but processing of the data will mainly be done in the office. This procedure can eliminate transcription errors and increase productivity. However, positional and topological errors in captured topographic features cannot be detected until features are formed manually in the office with the aid of a sketch from the field crew. In addition, mistakes can also be made because of incorrect interpretation of the diagram. Very often, revisit of the site is necessary to solve the problems or recapture the missing data. Survey data captured will be assigned to an object and associated attribute information can be assigned. Information gathered from this system can easily be loaded into a GIS for assessment or updating as we made in this project.

4.2 CONTROL POINT SURVEYING

By definition, a control survey consists of determining the horizontal and vertical or spatial positions of arbitrarily located points. Traditionally, horizontal and vertical controls have been established separately, but with the advent of GPS, which provides both horizontal and vertical control in the same operation, this separation is rapidly disappearing.

A geodetic control survey takes into account the shape of the earth and generally is used for primary control networks of large extent and high precision, such as those surveys established for continents, states, and counties. The bulk of the geodetic surveys performed currently are done with GPS for the horizontal positions but geodetic leveling still is used for precise vertical control. By virtue of the characteristics of the system and the reduction process. Differential GPS automatically yields a geodetic horizontal survey. [Source: Reference No.2]

An engineering control survey provides the horizontal and vertical control for the design and construction of private and public works. Depending on the size and scope of the project, such a survey may be geodetic but often is simply a plane survey for horizontal control with precise or differential levelling for vertical control. Ideally, the engineering survey should originate and close on horizontal and vertical control points in the national or state geodetic network.

The distinguishing feature of a topographic survey is the determination of the location, both in plan and elevation, of selected ground points that are necessary for plotting contour lines and the planimetric location of features on the topographic map.

4.2.1 Type of control point

Control consists of two parts:

1. *Horizontal control,* in which the planimetric positions of specific control points are located.
2. *Vertical control,* in which elevations are established on specified benchmarks located throughout the area to be mapped.

4.2.1.1 Horizontal Ground Control Points

Horizontal ground control points as they are usually defined are points whose horizontal positions are known relative to a ground coordinate system, may be arbitrary or real coordinate system such as "Palestinian Coordinate System". Horizontal position of a point means that coordinates (X, Y) or (E, N) of point are known with respect to horizontal datum "origin".

GPS, total station system traverse, aerial photogrammetric methods, ordinary traverse, or Trilateration and triangulation can establish horizontal control. Frequently, a combination of certain of these methods is used.

Horizontal control determination by aerial photogrammetric methods is feasible and particularly applicable to small-scale mapping of large areas. Note that traditional photogrammetric control surveys require a basic framework of horizontal control points established by GPS or total station traverse. However, if a GPS receiver is used aboard the aircraft procuring the aerial photography, the number of ground control points can be substantially reduced, although as yet, not eliminated.

4.2.1.2 Vertical Ground Control Points

The purpose of vertical control is to establish benchmarks at convenient intervals over the area to serve:

1. As points of departure and closure for operations of topographic parties when locating details.
2. As reference marks during subsequent construction work.

Vertical control usually is accomplished by direct differential levelling, but for small areas or in rough country the vertical control is frequently established by trigonometric levelling.

Standard of accuracy for levelling

- The FGCS (Bossler, 1984) also sets forth vertical network standard based on an elevation different accuracy.
- Designated by b, where $b = ¢/\sqrt{d}$ ………………………………………..(4.1)

  In which, *d* = approximate horizontal distance between control point position, in Km.

  ¢ = Estimated standard deviation of elevation different, in mm, between control points propagated from a least square an adjustment.

The unit of b are in mm/Km. Elevation different accuracy values for b are given for the various order of levelling in table 4.1

Table 4.1: Elevation accuracy standards

CLASSIFICATION	**MAX ELEVATION DIFFERENT ACCURACY,** b
First order, class I	0.5
First order, class II	0.7
Second order, class I	1.0
Second order, class II	1.3
Third order	2.0

4.2.2 Well-Defined Points

A well-defined point represents a feature for which the horizontal position is known to a high degree of accuracy and position with respect to the geodetic datum. For the purpose of accuracy testing, well-defined points must be easily visible or recoverable on the ground, on the independent source of higher accuracy, and on the product itself. Graphic contour data and digital hypsographic data may not contain well-defined points. The selected points will differ depending on the type of dataset and output scale of the dataset. [Source: Reference No.10.a]

For graphic maps and vector data, suitable well-defined points represent right-angle intersections of roads, railroads, or other linear mapped features, such as canals, ditches, trails, fence lines, and pipelines. For orthoimagery, suitable well-defined points may represent features such as small isolated shrubs or bushes, in addition to right-angle intersections of linear features. For map products at scales of 1:5,000 or larger, such as engineering plats or property maps, suitable well-defined points may represent additional features such as utility access covers and intersections of sidewalks, curbs, or gutters.

4.2.3 Data acquisition for the independent source of higher accuracy

The independent source of higher accuracy shall be acquired separately from data used in the aerotriangulation solution or other production procedures. The independent source of higher accuracy shall be of the highest accuracy feasible and practicable to evaluate the accuracy of the dataset. [Source: Reference No.10.a]

Although guidelines given here are for geodetic ground surveys, the geodetic survey is only one of many possible ways to acquire data for the independent source of higher accuracy. Geodetic control surveys are designed and executed using field specifications for geodetic control surveys (Federal Geodetic Control Committee, 1984). Accuracy of geodetic control surveys is evaluated using Standards for Geodetic Networks (Federal Geographic Data Committee, 1998). To evaluate accuracy, if the accuracy of geodetic survey is sufficiently greater than the positional accuracy value given in the product specification, compare the FGCS network accuracy reported for the geodetic survey with the accuracy value given by the product specification for the dataset.

Other possible sources for higher accuracy information are Global Positioning System (GPS) ground surveys, photogrammetric methods, and databases of high accuracy point coordinates.

4.2.4 Check Point Location

Due to the diversity of user requirements for digital geospatial data and maps, it is not realistic to include statements in this standard that specify the spatial distribution of checkpoints. Data and/or map producers must determine checkpoint locations. This section provides guidelines for distributing the checkpoint locations.

Check points may be distributed more densely in the vicinity of important features and more sparsely in areas that are of little or no interest. When data exist for only a portion of the dataset, confine test points to that area. When the distribution of error is likely to be non-random, it may be desirable to locate checkpoints to correspond to the error distribution.

For a dataset covering a rectangular area that is believed to have uniform positional accuracy, check points may be distributed so that points are spaced at intervals of at least 10 percent of the diagonal distance across the dataset *and* at least 20 percent of the points are located in each quadrant of the dataset.

4.3 METHODS FOR HORIZONTAL POSITIONING

4.3.1 Intersection

When coordinate of point are given and the azimuth and distance to a second point also are known, it is possible to compute the coordinate of the second point. Similarly, if the coordinate are given for the two ends of a line directions are observed from each end of this line to a third point not on the line, then coordinates of that third point can be calculated. This procedure is called *location by intersection*.

4.3.2 Resection

When angles between lines to three points of known position are observed from a point unknown position, the coordinate of the unknown point can be calculated. This procedure is called *location by resection.*

4.3.3 Triangulation

Triangulation is a measurement system comprised of joined or overlapping triangles of angular observations supported by occasional distance and astronomic observations. Triangulation is used to extend horizontal control network geometry.

A triangulation system extending over a wide area likewise is divided into figure irregularly overlapping and intermingling. The computation for such a system can be arranged to provide checks on the computed values for most of the side. As many sides as possible are included in the route through which the computations are carried from one base line the next.

4.3.4 Trilateration

A Trilateration network consists of a system of joined or overlapping triangles or polygons in which all lengths are measured and only enough angles or directions are observed to establish azimuth.

If one considers a pure Trilateration network, in only distances are measured, the number of mathematical conditions in the figure must be formulated in term of these distances.

The new survey is required to tie to at least four network control points spaced well apart. These network points must have datum values equivalent to or better than the intended order (and class) of the new survey

4.3.5 Tringulateration

A Tringulateration network consists of a system based on measured distances and angles.

4.3.6 Traverse

Traverse surveying is one of the most commonly used methods for determining the relative positions of a number of survey points. While leveling is used to establish the elevations of points, traverse surveying is used to determine the horizontal coordinates of these points. Basically, it consists of repeated application of the method of locating by angle and distance. By starting from a point of known position and a line of known direction, the location of a new point is determined by measuring the distances and angle from the known point. Then, the location of another new point is determined by angle and distance measurement from the newly located point. This procedure is repeated from point to point. The resulting geometric figure is called a traverse.

4.3.6.1 Purpose of the traverse:

The traverse serves several purposes among, which are; Property survey establishes boundaries, location and construction layout surveys for highways, and railways and other works. Ground control surveys for photogrammetric mapping or map assessment. Map revision of new features like buildings, streets...etc.

4.3.6.2 Choice of traverse stations:

Traverse stations should be located so that:

Traverse lines should be as close as possible to the boundaries of the tract of land to be surveyed, Distances between stations should be approximately equal and the shortest line should be greater than one third of the longest line.

Stations should be located on firm ground. When standing on one station and using Total Station Instrument, it must be easy to see the back sight and foresight stations.

Horizontal angle between traverse lines should be neither sharp nor wide-open

4.3.6.3 Types of traverse:

There are two main types of traverse:

I. Open traverse:

This type originates at a point of known position and terminates at a point of unknown position (figure 4.1). To minimize errors, distances can be measured twice, angles repeated, and magnetic azimuth observed on all lines. This type is used in certain works such as locating the centreline of a tunnel during construction. For projects requiring high accuracy, it must not be used.

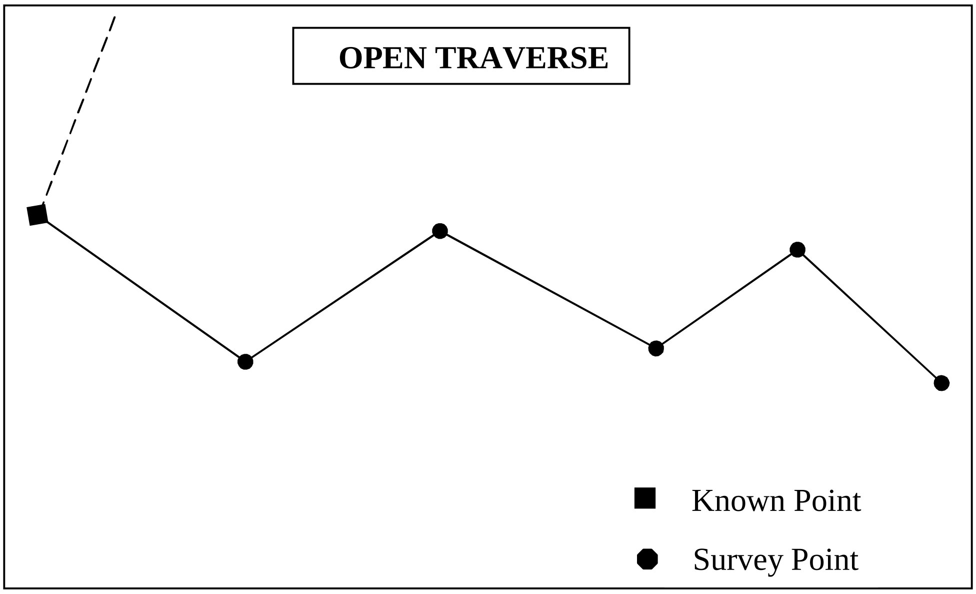


Figure (4.1): Open traverse

II. Close traverse:

This type originates at a point of known position and terminates at the same point or at another point of known position as we made in this project (link traverse) (figure 4.2). This type is preferred to the first type because it provides a check on errors.

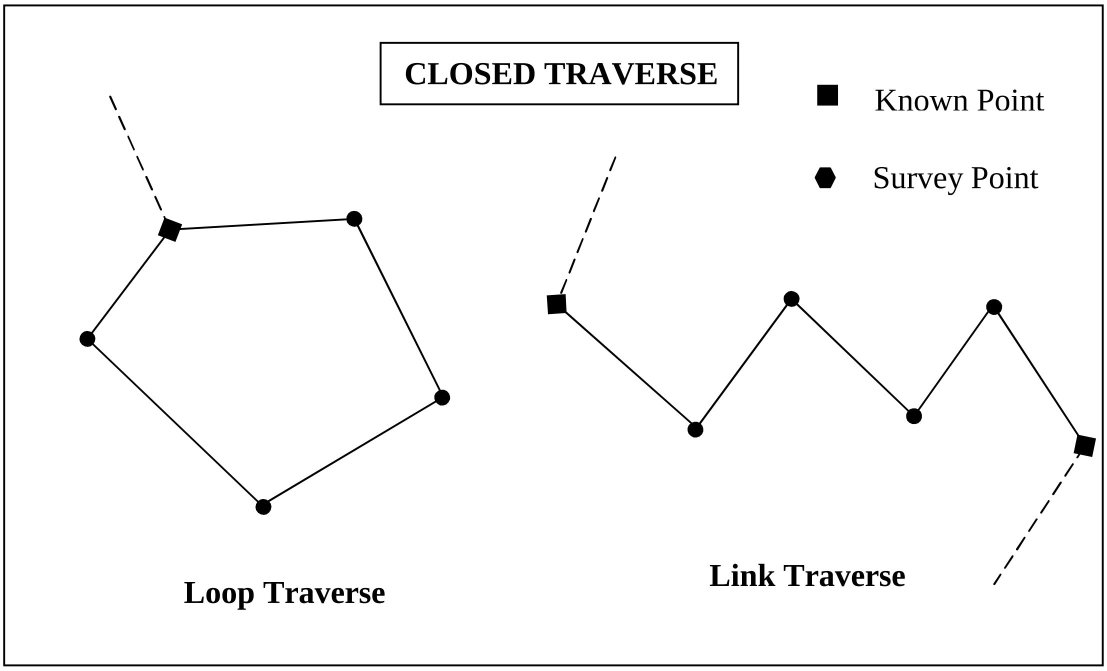


Figure (4.2): Closed traverse

Table 4.2: horizontal control specification for traverses (FGDC). [Source: Reference No.2]

ORDER CLASS	**FIRST**	**SECOND**		**THIRD**	
		I	**II**	**I**	**II**
Network geometry					
Station spacing not less than (km)	10	4	2	0.5	0.5
Maximum deviation of main traverse from straight-line	20°	20°	25°	30°	40°
Minimum number of benchmark ties	2	2	2	2	2

Benchmark tie spacing not more that (segments)	6	8	10	15	20
Astronomic azimuth spacing not more than (segments)	6	12	20	25	40
Minimum number of network control points	4	3	2	2	2
Instrumentation					
Theodolite, least count	0.2"	1.0"	1.0"	1.0"	1.0"
Field procedures					
Directions					
Number of positions	16	8 or 12*	6 or 8**	4	2
Standard deviation of mean not to exceed	0.4"	0.5"	0.8"	1.2"	2.0"
Rejection limit from the mean	4"	5"	5"	5"	5"
Reciprocal vertical angles (along distance sight path)					
Number of independent observations direct /reverse	3	3	2	2	2
Maximum spread	10"	10"	10"	10"	20"
Maximum time interval between reciprocal angles (hr)	1	1	1	1	1
Astronomic azimuths					
Observations per night	16	16	12	8	4
Number of nights	2	2	1	1	1
Standard deviation of mean not to exceed	0.45"	0.45"	0.6"	1.0"	1.7"

Rejection limit from the mean	5"	5"	5"	6"	6"
Infrared distances					
Minimum number of measurements	1	1	1	1	1
Minimum number of concentric observations /measurement	1	1	1	1	1
Maximum number of offset observations /measurement	1	1	1	-	-
Minimum difference from mean of observations (mm)	10	10	10	-	-
Minimum number of observation	10	10	10	10	10
Maximum difference from mean of readings (mm)	:	:	:	:	:
Microwave distances					
Minimum number of measurements	-	1	1	1	1
Minimum number of concentric observations /measurement	-	2[&]	1[&]	1[&]	1[&]
Maximum difference from mean of observations (mm)	-	150	150	200	200
Minimum number of readings /observation	-	20	20	10	10
Maximum difference from mean of readings (mm)	-	:	:	:	:

*8 if 0.2” 12if 1.0” resolution.

**6 if 0.2”. 8 if 1.0” resolution.

: As specified by manufacturer

& carried out at both ends of the line

. Only if decimal reading near 0 or high 9

4.3.6.4 Accuracy and specification for traverses

The accuracy of a traverse is judged into two separate processes: [Source: Reference No.4]

- Evaluating the local survey.
- Evaluating the network.

1. Evaluating the local survey:

Accuracies of a traverse on a local scale continue to be evaluated on the basis of ratio of misclosure (ROM) or relative positional accuracy, which is the ratio of the resultant error misclosure for the traverse to the total length of the traverse.

For example; if a traverse has a resultant closure error of 0.36m for a traverse having a total length of 1762 m. The ROM for this traverse is 0.36/1762 or 1 part in 4894.

Allowable azimuth and position closures for 1st, 2nd, and 3rd order traverses, should be cheeked by the federal geodetic control subcommittee (FGCS), in table (3.1).

The expression containing the square root is designed for larger lines, where higher proportional accuracy is required. Use the formula that gives the smallest permissible closure, the closure (e.g. 1: 00,000) is obtained by computing the difference between the computed and fixed values, and dividing this difference by K.

2. Evaluating Network control (horizontal control network standards):

When a horizontal control point is classified with a particular order and class, the National Geodetic Standards (NGS) certifies that the geodetic latitude and longitude of that control point bear a relation of specific a accuracy to the coordinates of all other points in the horizontal control network.

This relation is expressed, as distance accuracy 1:a. Distance accuracy is the ratio of the relative positional error of a pair of control point to the horizontal separation of those points.

Table 4.3 Distance accuracy standards

CLASSIFICATION	**MINIMUM DISTANCE ACCURACY, a**
First order	1: 100,000
Second order, class I	1: 50,000
Second order, class I I	1: 20,000
Third order, class I	1: 10.000
Third order, class I I	1: 5,000

Distance accuracy, 1: a is computed from minimally constrained, correctly weighted, least squared adjustment by:

$$a = d/s \quad \text{..........} \quad (4.2)$$

Where:

a= distance accuracy denominator

s= propagated standard deviation of distance between survey points obtained from the least squares adjustment
d=distance between survey point.

The distance accuracy pertains to all pairs of points (but in practice is computed for a sampling of a sampling of part of points). The worst distance accuracy (smallest denominator) is taken as the provisional accuracy. If this is substantially larger or smaller than the intended accuracy, then the provisional accuracy takes precedence.

4.3.6.5 Precise Traverses

Precise traverses are common among local surveyors for horizontal control extension, especially for small project. Fieldwork consists of two basic parts: reading horizontal angles at the traverses. Angles can be measured repeating or directional instrument, and distances measured with EDM equipment. [Source: Reference No.2]

Procedure for precise traverse computation vary, depending on whether geodetic or a plane reference system is used. There are two basic corrections for precise traverse:

1. Correction for distance

Reduction of slope distance at the ground to horizontal distance (chord) at elevation above the ellipsoid.

$$\overline{op} = \frac{\bar{o}p \sin ¥}{\sin(90+c)} \quad \text{..........} \quad (4.3)$$

Where;

$\overline{op}$ Horizontal distance at elevation above the ellipsoid, $\bar{o}p$ slope distance.

¥ $= 180^\circ - ($¤ $+ 90^\circ + c)$ ……………………………………...……………………….. (4.4)

¤ $= \frac{z_p + z_{o'}}{2}$ …………………………………………………………...………… (4.5)

$c = (16.192'' / km)(\bar{o}p_{km})$ "Defined curvature".

Reduction of the horizontal distance to horizontal distance at the ellipsoid (ellipsoidal distance).

$$QL = \frac{(R)(\overline{op})}{R + h_{o'}}$$ ……………………………………………………………………. (4.6)

QL Ellipsoid chord distance, R radius of the earth ellipsoid, $h_{o'}$ elevation of o' above the ellipsoid.

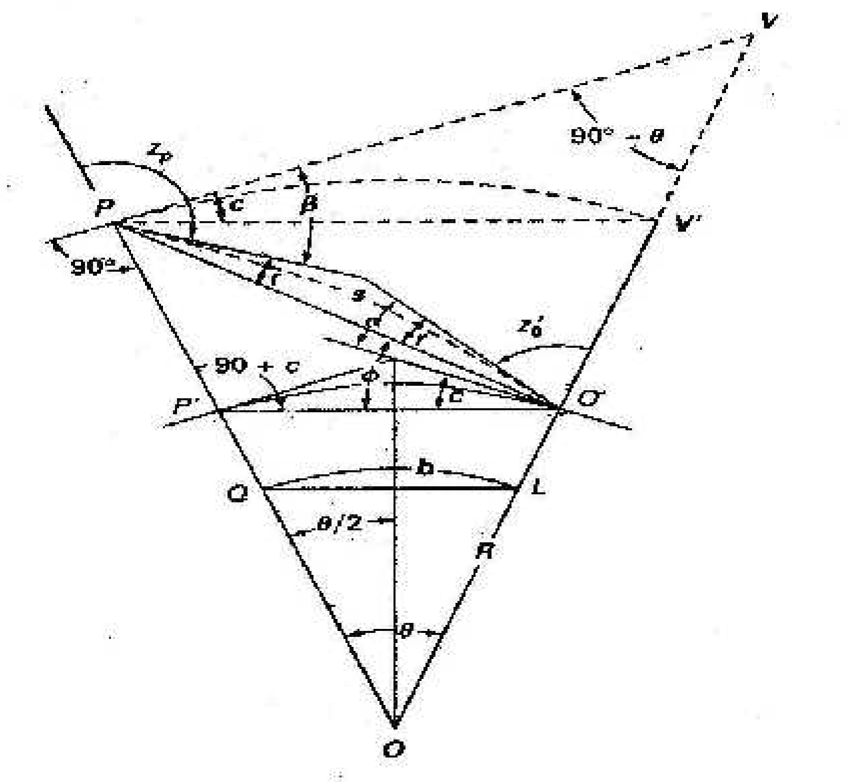
Compute the scale factor, which related to Casseni projection.

Or S.F = grid distance from the coordinate /measured distance.

Compute the grid distance by multiplying the ellipsoidal distance by scale factor.

$$G = (S.F)QL$$ …………………………………………………………………….(4.7)

Where G is the grid distance at the projection plane $S.F$ is the scale factor.



Figuer (4.3): reduction to sea – level distance by vertical angles

2. Reduction Azimuth to grid bearing

Geodetic azimuth can be transformed to grid bearing by the application of (t-T)$''$ correction.

(t-T)$''$ AB= $(2\Delta E_A + \Delta E_B)(N_{A-N_B})$ K…………………………….………..……......(4.8a)

(t-T)$''$ AC = $(2\Delta E_B + \Delta E_A)(N_{B-N_A})$ K…………………...…………………….……(4.8b)

$= \mp$ (t-T)$''$ AB- (t-T)$''$ AC…………………………………………………..(4.9)

Where

(t-T)$''$ AB = correction for line AB at station A.

(t-T)" AC = correction for line AC at station A.

= Computed angle from the grid coordinate.

–

= Observed angle.

ΔE = National grid easting – easting of the central meridian.

N = national grid northing.

K= constant ($845*10^{-6}$)

Easting of the central meridian = 500000m.

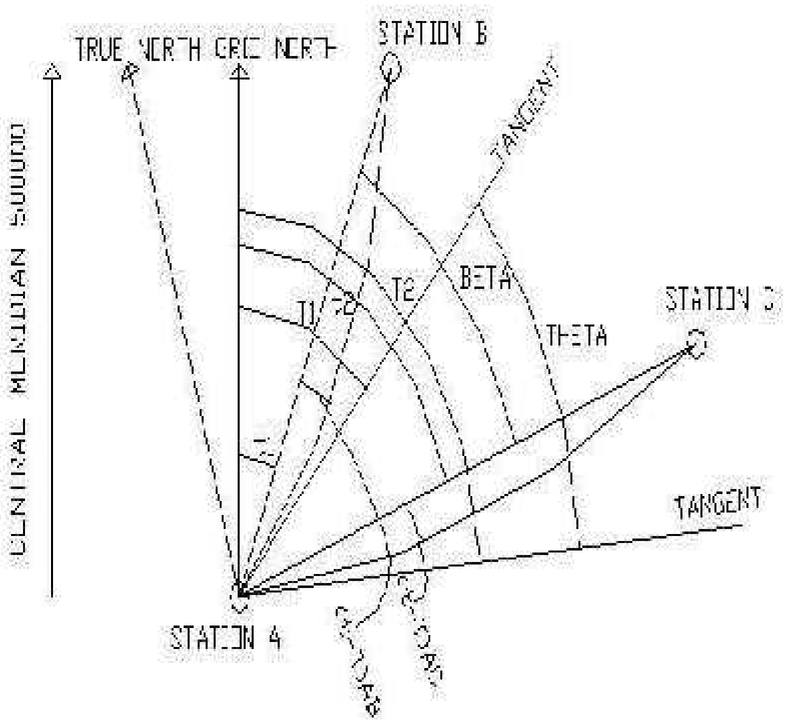


Figure (4.4): (t-T)" correction

4.4 ERROR PROPAGATION IN ANGLE AND DISTANCE MEASUREMENTS

Weather total station is used; errors are present in every horizontal angle measurement. Weather an instrument circles read, a small error is introduced in the final measured angle. Also, in pointing to a target, a small amount of error always occurs. Other major error sources in angle measurement include instrument and target setup error, and instrument levelling [Source: Reference No.6]. Each of these sources products random errors. They may be small or large, depending on the instrument, the operator, and the conditions at the time of the angle measurement. Increasing number of angle can reduce the effect of reading, pointing, and levelling errors. However, increasing sight distance can only reduce the effect of the instrument and target setup errors.

4.4.1 Error source in horizontal angles

Errors are present in every horizontal angle measurement, and there are explained as:

4.4.1.1 Reading Errors:

Errors in reading conventional transits and theodolites are dependent on the quality of the instruments optics, the size of the smallest division of the scale, and the observer's abilities.

Typical reading errors for a 1" micrometer theodolite can range from tenths of a second to several seconds.

Reading errors also occur with digital instrument, their size being dependent on the sensitivity of the particular electronic angular resolution system.

Typical values range from ± 1” for the precise instrument to ± 10” for the less expensive ones. These errors are random, and effect on an angle.

Angles Measured by the Repetition Method:

When measuring a horizontal angle by repetition method, the first occurs when the circle is zeroed, and the second when reading the final cumulative angle.

$$\pm\sim_{r} = \frac{\pm r\sqrt{2}}{n} \quad \text{……(4.10)}$$

Angles Measured by the Direction Method:

When a horizontal angle is measured by the directional method, the horizontal circle is read in both the back sight and foresight directions.

$$\pm_{\sim_r} = \frac{\pm_r\sqrt{2}}{\sqrt{n}} \quad \text{……(4.11)}$$

$\pm_{\sim_r}$: Estimated angular error due to when the directional method.

$\pm_r$: Observer reading error.

n: number of repetition.

4.4.1.2 Pointing errors

Accuracy in pointing to target is dependent on several factors. These include the optical qualities, target size, the observers personal, the weather conditions at time of observations.

$$\pm\sim_{p} = \frac{\pm p\sqrt{2}}{\sqrt{n}} \quad \text{……(4.12)}$$

Where;

$\pm\sim_p$: Estimated contribution to the overall angular error due to pointing.

$\pm_p$: Estimated errors in pointing, n = number of angle measurements.

4.4.1.3 Target-cantering errors

Whenever a target is set over a station, there will be some error due to faulty cantering. It can be attributed to environmental conditions, optical plummet errors, and plumb bob cantering error, personal abilities. The instrument usually within 0.000305 to 0.00305 m.

$$\pm\sim_t = \pm\frac{\sqrt{D_1^2 + D_2^2}}{D_1 D_2}\pm_t \circ \quad \text{...(4.13)}$$

Where $\pm\sim_t$ is the angular error due to target, D_1, D_2 are the distance from target to the instrument at station 1,2, respectively. $\circ = 206264.8$ “.

4.4.1.4 Instrument centering errors

Every time an instrument is centered over a point, there is some error in its position with respect to the true value station location. This error dependent on the quality of the instrument, the optical plummet, the quality of tripod, and the skill of the observer.

$$\pm\sim_i = \pm\frac{D_3}{D_1 D_2 \sqrt{2}}\pm_i \circ \quad \text{..(3.14)}$$

Where $\pm\sim_i$ error due to faulty instrument centering.

$$D_3^2 = D_1^2 + D_2^2 - D_1 D_2 \cos\sim$$

4.4.2 Errors in electronic distance measurements

$$\pm_D = \sqrt{\pm_i^2 + \pm_t^2 + a^2 + (D * b_{ppm})^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots(4.15)$$

Where $\pm_D$ is the error in the measured distance D, a and b are the instruments specified accuracy parameters.

4.5 ERROR ELLIPSES

The variance and standard deviation are measures of precision of the one-dimensional case of an angle or a distance.

In the 2-dimensional problems, such as the horizontal position of a point, error ellipses may be established around the point to designate precision regions of different probabilities.

The orientation of the ellipse relative to the x,y axes system fig (5.3) depends on the correlation between x and y. if they are uncorrelated, the ellipse axes will be parallel to x and y. If the two coordinates are equal precision, or $\pm x = \pm y$, the ellipse becomes a circle.

Consider the general case where the covariance matrix for the position of point (p) is given as:

$$\Sigma = \begin{bmatrix} \pm x^2 & \pm xy \\ \pm xy & \pm y^2 \end{bmatrix} \quad \ldots\ldots\ldots\ldots\ldots\ldots(4.16)$$

The semimajor and semiminor axes of the corresponding ellipse are computed in the following manner:

Compute the tangle by:

tan (2t) = $2\pm xy / \pm y^2 - \pm x^2$ ……………………………...…………………….(4.17)

Where t = $90 - ¤$

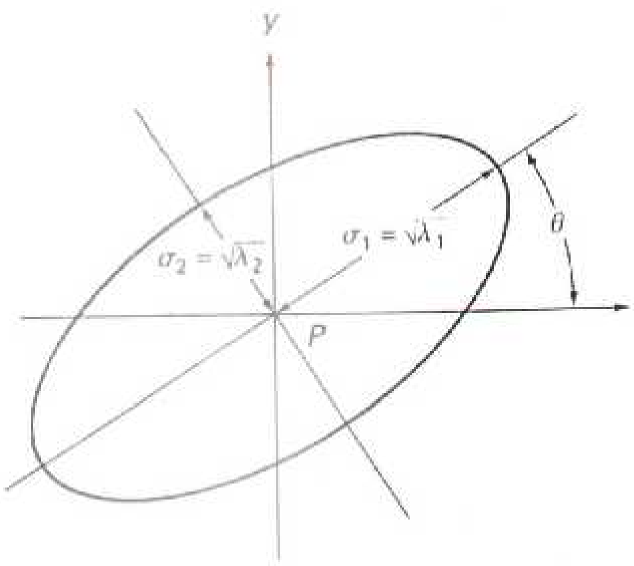


Figure (4.5): Error ellipse

The quadrant of 2t is determined according to the following signs in table 4.7:

Table 4.4: Algebraic sign

	QUADRANT	**NUMERATOR (**sin2t**)**	**DOMINATOR (**cos2t**)**
0	1	+	+
180	2	+	-
270	3	-	-
360	4	-	+

$\pm u^2 = \pm x^2 \sin^2(t) + 2\pm xy \cos(t)\sin(t) + \pm y^2 \cos^2(t)$ ………………………..…(4.18a)

$\pm v^2 = \pm x^2 \cos^2(t) - 2\pm xy \cos(t)\sin(t) + \pm y^2 \sin^2(t)$ …………………………..(4.18b)

The semimajor axis = $\pm u$ = a = $\sqrt{\pm u^2}$ …………………………………….……(4.19a)

The semiminor axis = $\pm v$ = b = $\sqrt{\pm v^2}$ …………………………………….……(4.19b)

The probability of falling on or inside the standard error ellipse is 0.394.

In manner similar to constructing intervals with given probabilities for the one-dimensional case, different size ellipses may be established, each with given probability. It should be obvious that the larger the size of the error ellipse, the larger is the probability.

Using the standard ellipse as abase, table (4.5) gives the scale multiplier k to enlarge the ellipse and the corresponding probability.

As an example, for an ellipse with axes a=2.4476$\pm u$ and b=2.447$\pm v$, where $\pm u$ and $\pm v$ are the semimajor axes and semiminor axes, respectively, of the standard ellipse, the probability that the point fall inside the ellipse is 0.95.

Table 4.5: Point estimate probability value [Source: Reference No.4]

2-DIMENSIONS		**3-DIMENSIONS**	
P	**K**	**P**	**K**
0.394	1.000	0.199	1.000
0.500	1.177	0.500	1.538
0.632	1.414	0.900	2.500
0.900	2.146	0.950	2.700
0.950	2.447	0.99	3.368
0.990	3.035	0.00	0.00

4.5.1 Evaluation the local accuracies for traverse (relative error ellipses).

In surveying, one frequently is interested in the relative accuracy between two points 1 and 2, in a horizontal network. Then, the coordinate differences are:

$$d_x = x_2 - x_1 \quad d_y = y_2 - y_1 \quad \text{.....(4.20)}$$

$$\Sigma_{1,2} = \begin{bmatrix} \pm^2 x1 & \pm x_1 y_1 & \pm x_1 x_2 & \pm x_1 y_2 \\ & \pm^2 y_1 & \pm y_1 x_1 & \pm x_2 y_2 \\ & & \pm^2 x_2 & \pm x_2 y_2 \\ symmetric & & & \pm y_2 \end{bmatrix} = JP1{,}2 or JX$$

Then,

$$d = \begin{bmatrix} d_x \\ d_y \end{bmatrix} = \begin{bmatrix} -1 & 0 & 1 & 0 \\ 0 & -1 & 0 & 1 \end{bmatrix} * \begin{bmatrix} x_1 \\ y_1 \\ x_2 \\ y_2 \end{bmatrix} = JP_{1,2} or JX$$

$$\Sigma dd = J * \Sigma_{1,2} * J^T \quad \text{.....(4.21)}$$

According to error propagation law from which,

$$\pm dx^2 = \pm x_1^2 - 2 \pm x_1 x_2 + \pm x_2^2 \quad \text{.....(4.22a)}$$

$$\pm dy^2 = \pm y_1^2 - 2 \pm y_1 y_2 + \pm y_2^2 \quad \text{.....(4.22b)}$$

$$\pm dxdy = \pm x_1 y_1 + \pm x_1 y_2 - \pm x_2 y_1 + \pm x_2 y_2 \quad \text{.....(4.22c)}$$

The variance, $\pm dy^2, \pm dy^2$ and covariance, $\pm dxdy$ of the line 1-2 have to be substituted into equations (4.22a), (4.22b), and (4.22c) to calculate the elements of relative error ellipse.

Although both absolute error ellipses (for points) and relative error ellipses (for lines) are used to evaluate adjustment quality, it is frequently more convenient to replace the two-dimensional representation by one-dimensional single quantity (similar to $\pm$). In this case, a circular probability distribution is substituted for the elliptical probability distribution.

Consequently, a single circular standard deviation $\pm c$ is calculated from the two semi axes of the relative magnitudes of these axes.

Let $\Sigma xx = \begin{bmatrix} \pm^2 x & \pm xy \\ \pm xy & \pm^2 y \end{bmatrix}$ represent the covariance matrix for the x,y coordinate of a point. Then, $\pm^2 u = a$ and $\pm v = b$ are the semimajor and semiminor axis of the error ellipse, respectively.

The ratio $\pm v / \pm u$ =b/a determines the relationship used to calculate $\pm c$ according to these cases:

1. *When* $\pm v / \pm u$ *is between 1.0 and 0.6, then.*

$\pm c = 0.5222 \pm v + 0.4778 \pm u$ …………………………………………………….(4.23a)

A good approximation that yields as lightly larger $\pm c$ (i.e. on the safe side) is given by:

$\pm c = 0.5(\pm v + \pm u) = 0.5(a + b)$ ………………………………………………….(4.23b)

2. *Which may be extended to the limit of* $\pm \min / \pm \max = \pm v / \pm u \le 0.2$

Instead of equation (4.23c) we can use:

$\pm \bar{c} = \left[0.5(\pm^2 c + \pm^2 y)\right]^{1/2}$ ……………………………………………...…… (4.23c)

This equation is applied only when $\pm v / \pm u$ is between 1.0 and 0.8, in which case it yields essentially the same value of $\pm c$ as in equation (4.23c).

As the ratio of $\pm v / \pm u$ decreases, $\pm c$ from equation (4.23b) gets progressively larger than that from equation (4.23c) with maximum increase of about 20% at $\pm v / \pm u = 0.2$.

Of course, the probability associated with standard error circle is the same as for the standard error ellipse, 0.394. The multipliers given in table (4.7) also still apply for circular errors of different probabilities.

4.5.2 Evaluation the network accuracies for traverse (standard error circle).

Consider the general case where the covariance matrix for the position of point (p) is given as:

$$\sum = \begin{bmatrix} \pm x^2 & \pm xy \\ \pm xy & \pm y^2 \end{bmatrix}$$

$$a^2 - (\pm^2 dx + \pm^2 dy) * a + (\pm^2 dx * \pm^2 dy - \pm^2 dxdy) = 0$$

Then a_1, a_2 eigenvalues of the covariance matrix.

Semi major axis a= $\sqrt{a_1}$

Semi minor axis b= $\sqrt{a_2}$

Compute ratio b/a to determine which equation is used to compute $\pm c$.

Scaling $\pm c$ to probability 63.2% by multiplying $\pm c$ with 1.414:

$\pm c_b = \pm c * 1.414$

Table 4.6: Accuracy standards for horizontal position, ellipsoid height, and orthometric height (modified courtesy of FGDC, 1997 [Source: Reference No.2]

ACCURACY CLASSIFICATION	**95% CONFIDENCE, m (less than or equal to)**
1 millimeter	0.001
2 millimeters	0.002
5 millimeters	0.005
1 centimeter	0.010
2 centimeters	0.020
5 centimeters	0.050
1 decimeter	0.100
2 decimeters	0.200
5 decimeters	0.500
1 meter	1.000
2 meters	2.000
5 meters	5.000
10 meters	10.000

MAPS

Maps may be divide into two classes: those that become a part of public records of land division and those that form the basis of studies for private and public works. The best examples of the former are the plans filed as parts of deeds in the county registry of deeds in most states of the United states. Good examples of the latter are the preliminary maps along the proposed route of a highway. It is evident that any dividing line between these two classes is indistinct, because many maps might serve both purposes.

Maps that form the basis for studies may be divided into two types : line- drawn maps, drawn automatically by CADD systems or by a draftsperson using data from field surveys and those compiled photogrammetrically using stereo pairs of vertical aerial photographs ,and orthophoto maps, also produced by photogrammetric methods from stereo -aerial coverage.

Maps are further divided into the following two general categories: planimetric maps, which graphically represent in plan such natural and artificially features as streams, lakes, boundaries , condition and culture of the land, and public and private works: and topographic maps, which include not only some or al of the preceding features but also represent the relief or contour of the ground.

Maps of large areas, such as a state or country, which show the locations of cities, towns, streams, lakes, and the boundary lines o principal civil divisions, are called geographic maps. Maps of this character, which show the general location lf some kind of the words of human beings, are designated by the name of the works represented. Thus we have a railroad map of the united states or an irrigation map of California. Maps of this type which emphasize a single topic and where the entire map is devoted to showing the geographic distribution or concentration of a specific subject, are called thematic maps.

Topographic maps show shorelines, locations and depths of soundings or lines of equal depth, bottom conditions, and sufficient planimetric or topographic features of lands adjacent to the shores to interrelate the positions of the surface with the underwater features. Charts of the U.S. National ocean survey are good examples of hydrographic maps. Such maps form an important part of the basic information required for the production of environmental impact statements.

Maps are constructed using data acquired in the field by yield survey methods, such as radial surveys with a total station system or kinematics GPS surveys . Information from these surveys, stored in electronic field data collectors supplemented by notes in conventional field notebooks, is electronically transferred or keyed into an electronic computer in the office for processing and plotting using ADD. Maps more frequently are carried by either aircraft or satellite, Note that, for a map compiled by photogrammetric a carried methods, a sufficient number of ground control points of known positions must be identifiable in the photographic or sensed record to allow scaling the map and selecting the proper datum.

For small jobs (e.g., lot surveys, land developments, and construction jobs), traditional methods and manual drawing still may be employed. From an educational standpoint, one cannot underestimate the value of the experience of manually plotting a map as it leads to a much - enhanced ability to use and evaluate properly the many features found in maps.

An increasing number of maps also is produced from map data obtained from a wide variety of sources. Data from old maps . from aerial photography, and data acquired by field methods can be digitized and tired in the memory of an electronic computer, on diskette, referred to as a database. The database so formed is called a digital map. When desired. These data can be retrieved and pltted automatically using a CADD plotting capability, then become an integral part of a larger electronic controlling system called a geographic or land information system. Use of a GIS or LIS permits the operator to retrieve and overlay maps with an extensive variety of other types of data such as utility maps. Land use portrayals, census information, or vehicle traffic distributions and provides an extremely powerful tool for the planning and design involved in a wide range of projects,

PRECISION OF MEASUREMENTS

In dealing with abstract quantities, which have become accustomed to thinking in terms of exact values, the student of surveying should appreciate that the physical measurements acquired in the process of surveying are correct only within certain limits because of errors that cannot be eliminated. The degree of precision of a given measurement depends on the methods and instruments employed and on other conditions surrounding the survey. It is desirable that all measurements be made with high precision, but unfortunately a given increase in precision usually is accompanied by more than a directly proportionate increase in the time and effort of the surveyor. It therefore becomes the duty of the surveyor to in the time and effort of the surveyor. It therefore becomes the duty of the surveyor to maintain as high a degree of precision as can be justified by the purpose of the surveyor but no higher. It is important, then , that the surveyor have a thorough knowledge of (1 th sources and types of errors. (2 the effect of errors on field measurements. (3 the instruments and methods to be employed ti keep the magnitude of the errors within allowable limits, and (4 the intended use of the survey data.

A completed discussion of error analysis as related to surveying procedure is presented

FIELD AND OFFICE WORK

GENERAL

The nature of surveying measurements already has been indicated. Much of the field and office work involved in the acquisition and processing of measurement is performed concurrently. Field and office work for a complete survey consists of

1. planning and design of the survey; adoption of specifications. Adoption of a map projection and coordinate system and of a proper datum, selection of equipment and procedures.

2.

LAND AND GEOGRAPHIC INFORMATION SYSTEMS

Land information systems (LISs) and geographic information systems (GLSs) are new areas of activity which have rapidly assumed positions of major prominence in surveying. These computer-based systems enable storing, integrating, manipulating, analyzing, and displaying virtually any type of spatially related information about our environment. LISs are being used at all levels of government, and by businesses, private industry, and public utilities to assist in management and decision making. Specific applications have occurred in many diverse areas and include natural resource management, facilities sitting and management, land records modernization, demographic and market analysis, emergency response and fleet operations, infrastructure management, and regional, national, and global environmental monitoring.

Data stored within LISs and GLSs may be both natural and cultural, and be derived from surveys, existing maps, charts, aerial and satellite photos, statistics, tabular data, and other documents. Specific types of information, or so-called layers, may include political boundaries, individual property ownership, population distribution, locations of natural resources. Transportation networks , utilities, zoning, hydrograph, soil types, gand use, vegetation types, wetlands, and many more.

An essential ingredient of all information entered into LIS and GLS databases is that it be spatially related, that is, located in a common geographic reference framework. only then are the different layers of information physically relatable so they can be analyzed using computers to support decision making. This geographic positional requirement will place a heavy demand rpon surveyors in the future, who will play key roles in designing, implementing, and managing these systems. Surveyors from virtually all of the specialized areas described in the preceding section will be involved in developing the needed databases. Their work will include establishing the required basic control frameworks, conducting boundary surveys and preparing legal descriptions of property ownership; performing topographic and hydrographic surveys by ground, aerial, and satellite methods; compiling and digitizing maps; and assembling a variety of other digital data files.

This subject seems appropriately covered at the end, after each of the other types of surveys needed to support these systems has been discussed.

FEDERAL SURVEYING AGENCIES

Several agencies of the U.S. government perform extensive surveying and mapping. Four of the major ones are:

1. The coast and Geodetic survey. Now the national geodetic survey (NGS) and part of the national ocean survey (NOS). was originally organized to map the coast. Its activities include control surveys, preparation of nautical and aeronautical charts, photogrammetric surveys, tide and current studies , collection of magnetic data, gravimetric surveys, and work wide control survey operations that involve satellites. The basic control points established by this organization are the foundation for all large area surveying. The NGS also plays a major role in coordinating and assisting in activities related to the development of modern LISs at local, state, and national levels.
2. The Bureau of land management (BLM) originally established in 1812 as the general land office, directs the public lands surveys. Lines and corners have been set for most public lands in the conterminous united states, but much work remains in Alaska and is proceeding with modern techniques.
3. The U.S Geological survey (USGS). Established in 1879, has the responsibility for preparing maps of the entire country. Its standard 7 ½ quadrangle maps show topographic and cultural features, and are suitable for general use as well as a variety of engineering and scientific purposes. Nearly 10 million copies are distributed each year. Currently the USGS is engaged in a comprehensive program to develop a national digital cartographic database, which will consist of map data in a computer- readable format.
4. The defense Mapping Agency (DMA) prepares maps and associated products, and provides services for the Department of defense and all land combat forces, it is divided into the following military mapping groups: aerospace center, defense mapping school, hydrographic center, inter-American geodetic survey, and topographic center. The DMA topographic center fulfills a key mission in an era when accurate mapping, charting, and geodesy products are essential to realize the complete potential of new weapons. Technological advances in

weaponry demand corresponding improvements in mapping, charting, and geodesy to obtain accuracies that were just dreams only a few years ago.

5. In addition to these four agencies, units of the corps of engineers, U.S army, have made extensive surveys for emergency and military purposes. Some of these surveys provide data for engineering projects, such as those connected with flood control. Extensive surveys have also been conducted for special purposes by nearly 40 other federal agencies, including the forest service, national park service, international oundary commission, bureau of reclamation, Tennessee valley authority, millissippi river commission, U.S lake survey, and department of transportation. Like wise, many cities, counties, and states have had extensive surveying programs, as hae various utilities.

THE SURVEYING PROFESSION

Land or boundary surveying is classified as a learned profession because the modern practitioner needs a wide background of technical training and experience and must exercise a considerable amount of independent judgment. Registered (licensed) professional surveyors must have a thorough knowledge of mathematics-particularly geometry and trigonometry, but also calculus: competence with computers: solid understanding of surveying theory, instruments, and methods in the areas of geodesy, photogrammetry, remote sensing, and cartography: some competence in economics (including office management), geography, geology, astronomy, and dendrology: and a familiarity with laws pertaining to land and boundaries. They should be knowledgeable in both field operations and office computations. Above all, they are governed by a professional code of ethics, and are expected to charge reasonable fees for their work.

The personal qualifications of surveyors are as important as their technical ability in dealing with the public. They must be pationt and tactful with clients and their sometimes hostile neighbors, few people are aware of the painstaking research of old records required before field work is started. Diligent, time-consuming effort may be needed to locate corers on nearby tracts for checking purposes as well as to find corners for the property in question.

Permission to trespass on private property or to cut obstructing tree branches and shrubbery must be obtained through a proper approach. Such privileges are not conveyed by a surveying license or by employment in a state highway department (but a court order can be secured if a landowner objects to necessary surveys).

All 50 states, Guam, and Puerto rico have registration laws for professional surveyors and engineers (as do the provinces of Canada). Some states presently have separate licensing boards for surveyors. In general , a surveyors license is required to make property surveys, but not for construction, topographic, or route work, unless boundary corners are set.

To qualify for registration as either a professional land surveyor (LS) or professional engineer (PE) it iss necessary to nave an appropriate college degree, although some states allow relevant experience in lieu of formal education. In addition, candidates must acquire two or more years of additional practical experience, and also pass a two day comprehensive written examination. In most states a common national examination covering fundamentals and principles and practice lf land surveying is now used. However, two hours of the exam are devoted to local legal customs and aspects. Thus transfer of registration from one state to another has become easier.

Some states also require continuing education units (CEUs) for registration renewal, and many more are considering legislation that would ass this requirement.

CHAPTER 5

FIELDWORKS AND COMPUTATIONS

5.1 OVERVIEW

The nature of surveying projects in general contains fieldwork followed by observations analysis and assessment process "office work". This project includes several operations that were made officially starting with block map preparation, choose the traverses path, raw field observations entrance to the computer, cad drawing, maps assessment...etc as discussed in details in this chapter.

5.2 EXPLORATION OF STUDY AREA

An exploration visit to the study area was carried out to identify the boundary of the study area, and the features as houses, roads, and road intersections. This will help in suggesting possible lines of traverse, locations of control points and to identify the new features that are not been found in the current municipal topographic map.

The differences between the municipal topographic map of the area with scale 1:2500, which, received from Hebron Municipality, and the reality were identified. Also, the location of possible line traverse and location of control point were located on the hard-copy municipal topographic map and study area.

5.3 CONSTRUCTING TRAVERSE LINES

A traverse consists of straight lines connecting successive established points a long the route of a survey. In order to construct a net of reference of control points in the study area to check map registry and features locations of the municipal topographic map.

5.4 STANDARDS AND ACCURACY

In order to produce the work in good and acceptable conditions, it must be adjusted by quality control to meet the global or local standards and specifications. As this project is one of many projects that consists of observations and measurements, it must be controlled to meet these standards and specifications "i.e. to have errors less than the allowable errors".

The Palestinian Department of Surveying at Ministry of Housing allows the following errors in traverse: [Source: Reference No.6, page 148]

Table 5.1: Allowable errors in closed traverse

	ALLOWABLE ERROR	
Measurement Type	**Important areas (Urban areas)**	**Less important areas (Rural areas)**
Measured distances (m)	$\Delta L=0.0005L+0.03$	$\Delta L=0.0007L+0.03$
Angle closure error (sec)	$\Delta = 60''\sqrt{n}$	$\Delta = 90''\sqrt{n}$
Linear closure error (m)	$E=0.0006\,\Sigma L$	$\varepsilon=0.0009\,\Sigma L+0.20$

	+0.20	
Where L = measured length (m). Δ= Angle closure error. ΔL = difference between two measurements of one line. n = number of measured angles. $\varepsilon = \sqrt{£_y^2 + £_x^2}$, calculated linear closure error (m).		

5.5 MEASUREMENTS OF FIELD SURVEY

The traverse, starting at two control point, Shaheen Mosque (M), AL-Ahli hospital (A), and passing through Station B, C, D, E, and ending at two control point Zahdi house (F) Nonger (N), figure (5.1).

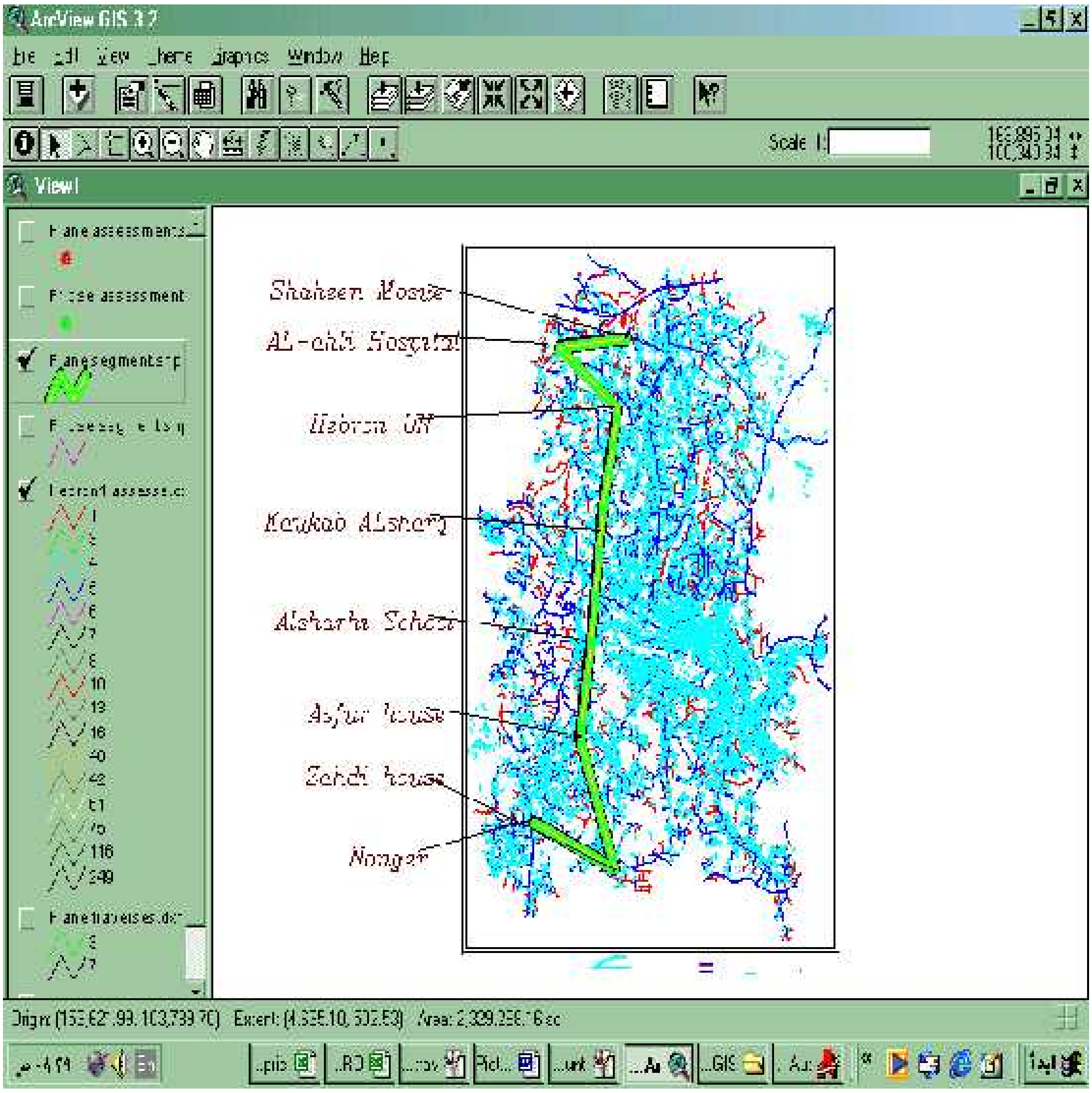


Figure (5.1): The route of the traverse

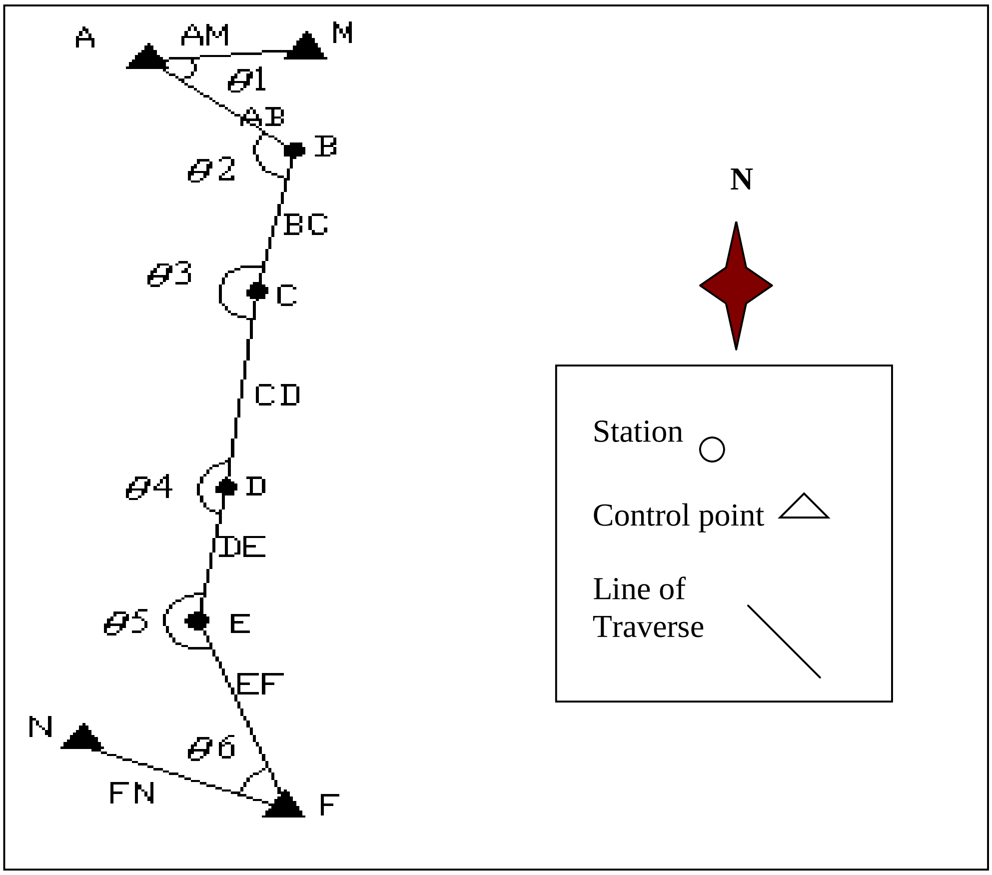


Figure (5.2): The link traverse showing traverse lines and its direction

5.5.1 Field survey measurements of traverses

A link traverse begins in two control points and ends on a different two control points. Normally, they were used to establish the positions of intermediate stations, as in B through E of the figure (5.2). The coordinates of the end points, stations M, A, F, and N of the figure, are known. Angular and linear misclosures are also computed for these types of traverse, and the resulting values used as the basis for the accepting or rejecting the measurements.

Table 5.2: Distance observations

FROM	**TO**	**DISTANCE (m)**	**STANDARD DEVIATION (m)** *(COMPUTED)*
M	A	1021.471	0.00
A	B	1136.181	0.845
B	C	1145.345	0.85
C	D	1575.170	0.76
D	E	1076.160	0.79
E	F	1605.447	0.78
F	N	1356.937	0.00

Table 5.3: Angle observations

BACK SIGHT	**OCCUPIED**	**FORESIGHT**	**ANGLE** ° ' "	**STANDARD DEVIATION** *(COMPUTED)*
M	A	B	44 13 33	2.2047
A	B	C	242 30 32.34	2.2055
B	C	D	175 23 04.67	2.1729
C	D	E	182 32 11	2.1849
D	E	F	150 49 14	2.1798
E	F	N	312 28 57	2.1294

Table 5.4: Control Points

STATION	**X (M)**	**Y (M)**
M	107256.16	158950.33
A	107161.35	157933.10

F	101189.49	158775.68
N	101711.63	157523.07

Table 5.5: Azimuth observations

FROM	TO	AZIMUTH ° ' "	STANDARD DEVIATION *(COMPUTED)*
M	A	264 40 30.6	0.00
B	C	128 54 03.6	2.205
C	D	191 24 35.9	3.118
D	E	189 19 51.6	4.324
E	F	160 09 05.61	11.896
F	N	292 38 21.61	5.339

5.5.2 Error propagation in angles and distances measurements

Example 1: Primary designed standards deviations for the first angle and distance measured on the maps, and the total error in angle.

1. Error propagation for the first angle

õ **Reading Error** of $2.5''$, an angle is read 6 times, as in equation (4.10)

$$\pm_{\sim R} = \pm \frac{\pm_r * \sqrt{2}}{\sqrt{n}}$$

$$\pm_{\sim R} = \pm \frac{2.5 * \sqrt{2}}{\sqrt{6}} = \pm 1.44$$

õ **Pointing Error *of*** $2.5''$, an angle is read 6 times, as in equation (4.12)

$$\pm_{\sim P} = \pm \frac{\pm_{p} * \sqrt{2}}{\sqrt{n}}$$

$$\pm_{\sim P} = \pm \frac{2.5 * \sqrt{2}}{\sqrt{6}} = \pm 1.44$$

õ **Target centering error*:*** Target centering over a station A is 0.00305m, the back sight and foresight distance is 1021.471m and 1136.181m, respectively, $° = 206264.8'' / rad$, as in the equation (4.13).

$$\pm_{\sim T} = \pm \frac{\sqrt{D^2{}_{1+} D^2{}_2}}{D_1 * D_2} * \pm_t * °$$

$$\pm_{\sim T} = \pm \frac{\sqrt{101.471^2{}_{+} 113.181^2}}{1021.471 * 1136.181} * 0.00305 * 206264.8 = \pm 0.83$$

õ **Instrument centering error*:*** Target centering over a station A is 0.00305m, the back sight and foresight distance is 1021.471m and 1136.181m, respectively, $° = 206264.8'' / rad$, as in the equation (4.14).

$$\pm_{\sim I} = \pm \frac{D_3}{D_1 * D_2 * \sqrt{2}} * \pm_i * °$$

$$D_3 = \sqrt{D^2{}_1 + D^2{}_2 - 2 * D_1 * D_2 * \cos \sim}$$

$$\pm_{\sim I} = \pm \frac{1225.823}{1021.471 * 1136.181 * \sqrt{2}} * 0.0035 * 206264.8 = \pm 0.16$$

Total error for the first angle:

$$\pm_{\sim Total} = \pm \sqrt{\pm^2{}_{\sim R} + \pm^2{}_{\sim P} + \pm^2{}_{\sim T} + \pm^2{}_{\sim I}}$$

$$\pm_{\sim Total} = \pm \sqrt{1.44^2 + 1.44^2 + 0.83^2 + 0.16^2} = \pm 2.20$$

2. **Errors for the first distance:** $a = 0.003m, b = 0.002m/km$

$$\pm_{dTotal} = \pm\sqrt{\pm^2{}_I + \pm^2{}_T + a^2 + (d * b_{ppm})^2}$$

$$\pm_{dTotal} = \pm\sqrt{0.16^2 + 0.83^2 + .003^2 + (1136.181 * 0.002_{ppm})^2} = \pm 0.84$$

Table 5.6: Primary designed standards deviations for angles and distances measured on the maps.

ANGLES	~r	~p	~t	~i
¯1	± 1.44"	± 1.44"	± 0.83"	± 0.16"
¯2	± 1.44"	± 1.44"	± 0.78"	± 0.33"
¯3	± 1.44"	± 1.44"	± 0.68"	± 0.34"
¯4	± 1.44"	± 1.44"	± 0.71"	± 0.35"
¯5	± 1.44"	± 1.44"	± 0.70"	± 0.33"
¯6	± 1.44"	± 1.44"	± 0.61	± 0.12"

Table 5.7: Estimated error in each angle and distance

ANGLES	±~	DISTANCES	±d
¯1	± 2.20	AB	± 0.84
¯2	± 2.21	BC	± 0.85
¯3	± 2.17	CD	± 0.76
¯4	± 2.18	DE	± 0.79
¯5	± 2.18	EF	± 0.78
¯6	± 2.13	FN	± 0.62

5.5.3 Angular misclosure

The actual angular error of closure can be calculated by two ways:

1. *By finding all the line Azimuths* until reaching to the final fixed line, and by subtracting the calculated Azimuth of the last line from its fixed Azimuth, error of closure can be determined.

   E = Final calculated Azimuth - Final Fixed Azimuth.

2. *By applying the rule that is:*

$$E = Az_i + \sum Angles - [(n-1)(180)] - Az_f \quad \text{.......................................} \quad (5.1)$$

Where; n = number of the angle to the right

Az_i = Initial fixed Azimuth.

Az_f = Final fixed Azimuth.

Azimuth of line from its end coordinates:

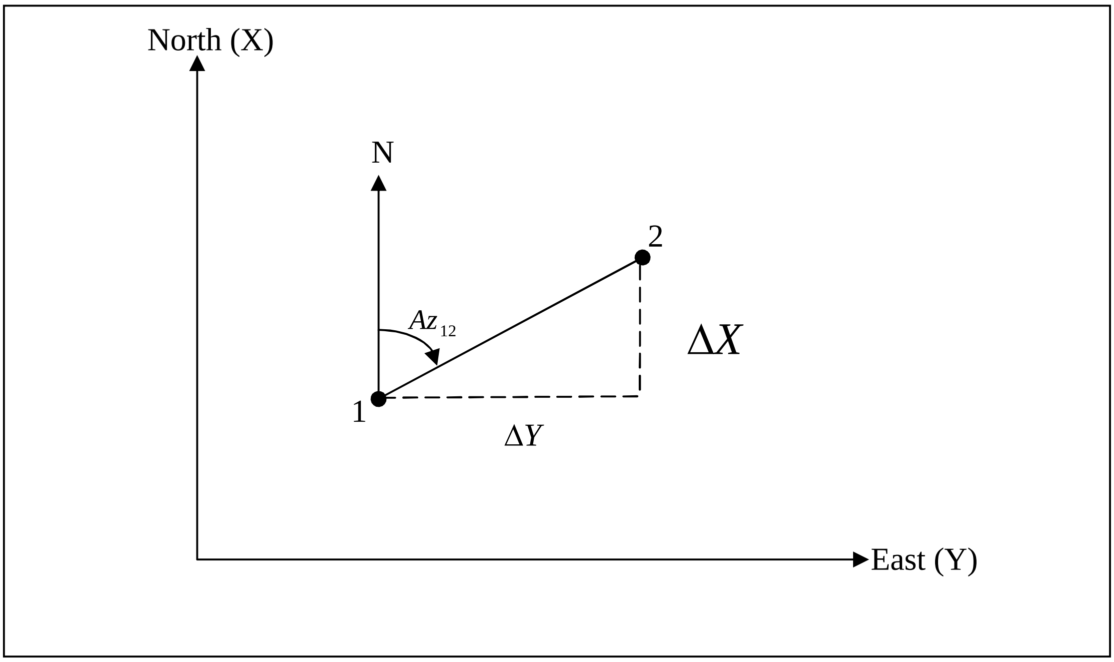


Figure (5.3): Azimuth of the line (1-2)

The whole circler Bearing (RB) of any line can be found by:

$$RB_{12} = \tan^{-1}\left(\frac{\Delta Y}{\Delta X}\right) = \tan^{-1}\left(\frac{Y_2 - Y_1}{X_2 - X_1}\right) \qquad (5.2)$$

Notation: in applying equation (5.2), you must determine which surveying quadrant the case meets, for example:

If $\left(\frac{\Delta Y}{\Delta X}\right) = \left(\frac{+}{+}\right) = \text{I}$ or $\left(\frac{\Delta Y}{\Delta X}\right) = \left(\frac{+}{-}\right) = \text{II}$

And this is illustrated in figure (5.4).

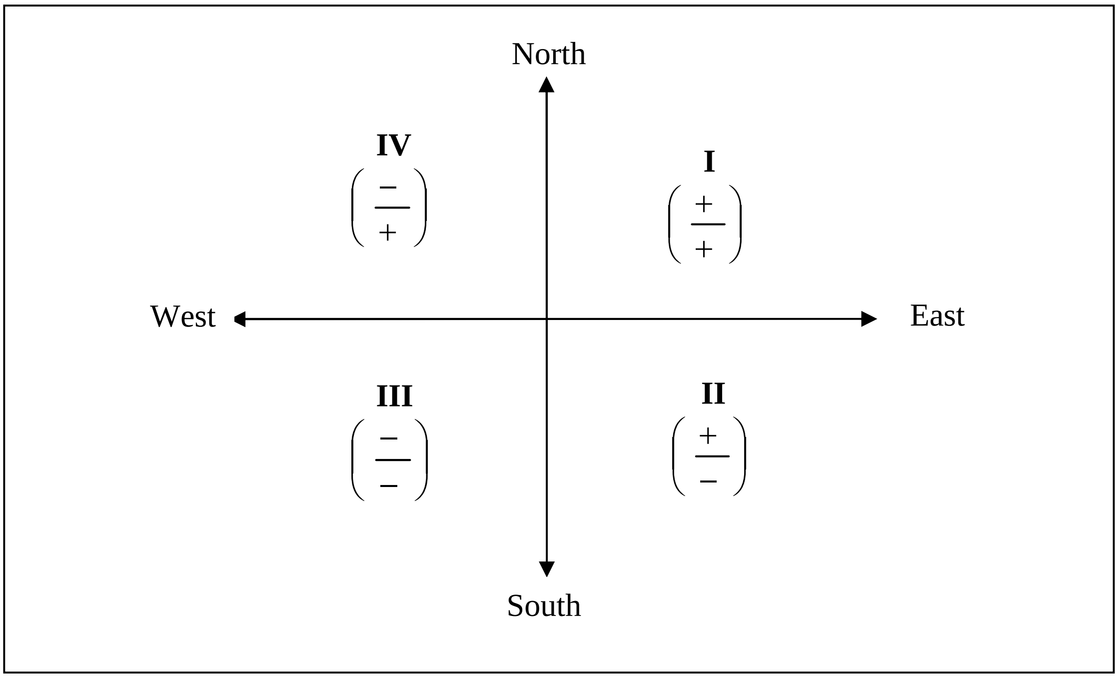


Figure (5.4): Four surveying quadrants with four cases

In order to find the Azimuth of the line, the following roles are applied:

If (R.B) in the first quadrant (I), then: Azimuth = (R.B).

If (R.B) in the second quadrant (II), then: Azimuth = 180 – (R.B).

If (R.B) in the third quadrant (III), then: Azimuth = 180 + (R.B).

If (R.B) in the forth quadrant (IV), then: Azimuth = 360 - (R.B).

3. *Azimuth to the next line:*

First, calculate the back Azimuth of the previous line:

Let the back Azimuth of the line is (Az_{2-1}), and the forward Azimuth of the same line is (Az_{1-2}) then:

If (Az_{1-2}) > 180, then: $Az_{2-1} = Az_{1-2}$ - 180.

If (Az_{1-2}) < 180, then: $Az_{2-1} = Az_{1-2}$ + 180.

Now add angle to the right to the Back Azimuth:

Sum= (Az_{2-1}) + Angle to the right.

If Sum > 360, then: $Az_{2-3} = Az_{2-1}$ + Angle to the right - 360

If Sum < 0, then: $Az_{2-3} = Az_{2-1}$ + Angle to the right + 360

If 0 < Sum < 360, then $Az_{2-3} = Az_{2-1}$ + Angle to the right

Where,

Az_{2-3}: The wanted Azimuth of the next line.

The estimated angular misclosure can be calculated after computing the estimated errors in the computed azimuths.

Estimated error of computed azimuths:

$$\pm_{AZc} = \sqrt{\pm^2_{AZp} + \pm^-_i} \quad \text{(5.3)}$$

$Az_c =$ Current Azimuth.

$Az_P =$ Previous Azimuth.

Estimated angular misclosure (standard error)

$$\pm_{estimated} = \sqrt{\pm^2_{fixed_{Azimuth}} + \pm^2_{actual_{Azimuth}}} \quad \text{(5.4)}$$

Expected (estimated) angular misclosure at confidence level 98%.

$\pm_{\exp ected 98\%} = t_{(\sim/2, ³)} * \pm_{estimated}$

$\sim / 2 = (1 - 95\%) / 2$

³ = Degree of freedom

$t_{(\sim/2, ³)} =$ Value from table of t-distribution

We accept angular misclosure at confidence of level 98%.

Determine the order and class of the traverse.

Actual Angular misclosure = constant * $\sqrt{n}$

N: number traverse segments.

Constant: depending on its value and according to the table of traverse standards. We determine the order and class of the traverse.

Actual angular misclosure = 292° 38' 2.61" – 292° 37' 42.2"

= 00° 00' 20.41"

Estimate standard error = $\sqrt{(0.0)^2 + (5.3388)^2} = \pm 5.3388$

20.41" = $constant \sqrt{N}$

$constant = 20.41'' / \sqrt{5} = 9.1276$

Third order ***class I is*** $10\sqrt{N}$ **,** Azimuth closure at Azimuth checkpoint. Here we have $9\sqrt{N}$ Azimuth closer.

5.5.4 Linear misclosure

The actual angular error of closure can be calculated by finding all the line latitudes (lat) and departures (dep) until reaching to the final fixed line, and then subtracting the (x, y) coordinates ($\Delta x, \Delta y$) of first and last control point of the traverse, and then compute ($\Delta lat, \Delta dep$) by subtracting $\Delta x, \Delta y$ from sum of latitudes and sum of departures respectively, and then find actual linear misclosure (Lc) which equal square root of ($\Delta^2 lat, \Delta^2 dep$).

$\Delta x =$ X final control –X first control

Δy = Y final control –Y first control.

$\Delta dep = \sum dep - \Delta y$ …………………………………...……….……….. (5.5a)

$\Delta lat = \sum lat - \Delta x$ …………………………………….……………………..(5.5b)

$L_c = \sqrt{(\Delta^2 lat + \Delta^2 dep)}$…………………………...…………………..… (5.6)

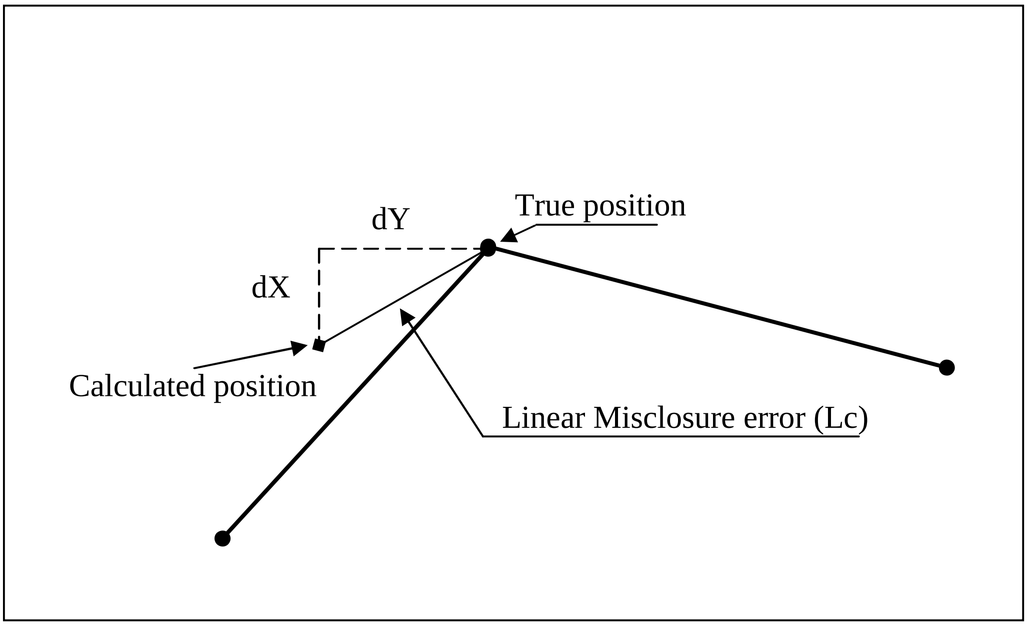


Figure (5.5): Resultant Linear Misclosure Error in closed traverse (Lc)

Latitude (Lat) and Departure (Dep) of any line can be found by:

$$\begin{aligned} Lat &= Dis\tan ce * \sin Az \\ Dep &= Dis\tan ce * \cos Az \end{aligned} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.7)$$

The estimated linear misclosure can be calculated using general of propagation

$$\Sigma_{LC} = A * \Sigma_{Lat,Dep} * A^T \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.8)$$

$$A = \begin{bmatrix} \cos(Az_{1A}) & -D\sin(Az_{1A}) & 0 & 0 & 0 & 0 \\ \sin(Az_{1A}) & D\cos(Az_{1A}) & 0 & 0 & 0 & 0 \\ 0 & 0 & \cos(Az_{AB}) & -D\sin(Az_{AB}) & 0 & 0 \\ 0 & 0 & \sin(Az_{AB}) & D\cos(Az_{AB}) & 0 & 0 \\ . & . & . & . & . & . \\ . & . & . & . & . & . \\ . & . & . & . & . & . \\ 0 & 0 & 0 & 0 & \cos(Az_{34}) & -D\sin(Az_{34}) \\ 0 & 0 & 0 & 0 & \sin(Az_{34}) & D\cos(Az_{34}) \end{bmatrix}$$

$$\Sigma_{LC} = \begin{bmatrix} \pm^2_{D1A} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \left(\frac{\pm_{Az1A}}{\circ}\right)^2 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & \pm^2_{DAB} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & \left(\frac{\pm_{AzAB}}{\circ}\right)^2 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & \pm^2_{DBC} & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & \left(\frac{\pm_{AzBC}}{\circ}\right)^2 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & \pm^2_{DCD} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & \left(\frac{\pm_{AzCD}}{\circ}\right)^2 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \pm^2_{DD3} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \left(\frac{\pm_{AzD3}}{\circ}\right)^2 \end{bmatrix}$$

Expected (estimated) linear misclosure at confidence level 98%.

$\pm_{expected_{98\%}} = t_{(\sim/2,\text{³})} * \Sigma_{LC}$

$\sim/2 = (1-95\%)/2$

³ = Degree of freedom

$t_{(\sim/2,\text{³})}$ = The value from table of t-distribution.

Table 5.8: latitudes and departures for plane traverses

COURSE	LATITUDE (M)	DEPARTURE (M)
A-B	-713.495	884.212
B-C	-1122.709	-226.581
C-D	-1564.107	-186.3593
D-E	-1061.9204	-174.4851
E-F	-1510.0758	545.1034
SUM	*-5972.3072*	*841.889*

$\Delta x =$ X F–X A = 101189.49 – 107161.35 = -5971.86 m.

$\Delta y =$ Y F –Y A = 158775.68 – 157933.10 = 842.58 m.

$\Delta dep = \sum dep - \Delta y$ = 842.066 - 842.58 = -0.693 m.

$\Delta lat = \sum lat - \Delta x$ = -5972.3072 - - 5971.86 = -0.447 m.

$L_c = \sqrt{(\Delta^2 lat + \Delta^2 dep})= \sqrt{-0.514^2 + -0.4472^2} = 0.825$

The estimated linear misclosure $= \Sigma_{LC} = A * \Sigma_{Lat,Dep} * A^T$

= 0.9889896

Expected (estimated) angular misclosure at confidence level 98%.

$\pm_{expected_{98\%}} = 4.541 * \sqrt{0.9889896} = \pm 4.516$

Relative precision = actual linear misclosure / Total length of traverse

$$= 0.852 / 6538.305 = 1 / 7925.22 \approx 1 / 8000$$

If actual linear misclosure $\leq$ estimated linear misclosure, then there is no reason to believe that there is a blunder.

If Relative precision = 1 / 1000, then the order is Third order class 1. Here we have 1 / 8000 position closure.

5.5.5 Determinations the order and class of the traverse

Relative precision = actual linear misclosure /Total length of traverse, depending on the value of relative precision and according to the table (4.3) of traverse standards. We determine the order and class of the traverse, from the data above the order of our traverses is Third order class I.

5.6 COORDINATES COMPUTATION

Then the coordinates of any point can be calculated by these rules:

X second point = X first point + latitude

Y second point = Y first point + departure

5.6.1 least square traverse adjustment

Calculate initial approximates for the unknown station coordinates.

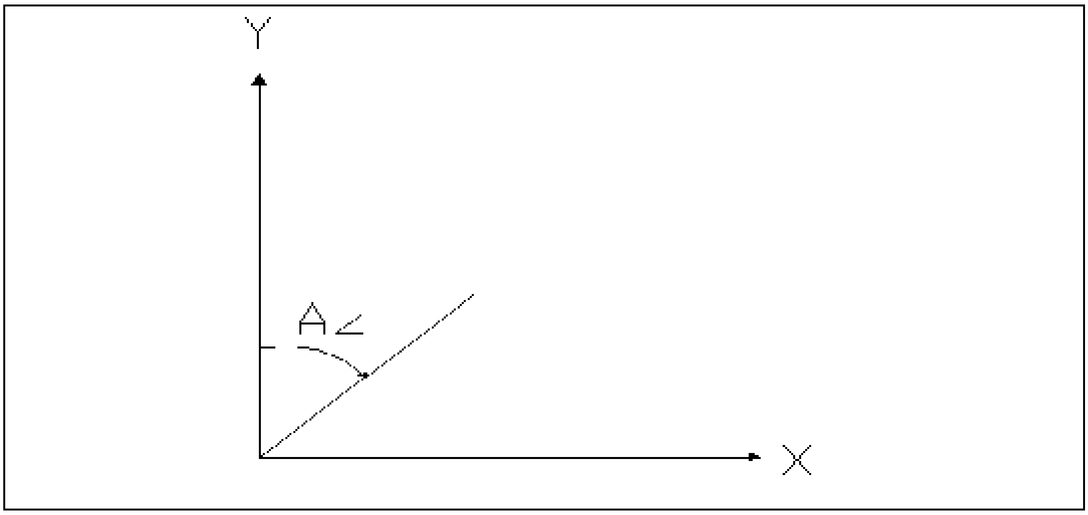


Figure (5.6): initial coordinate

X second point = X first point + distance * cos Az

Y second point = Y first point + distance * sin Az

Formulate the X and K matrices. The traverse in this problem contains four unknown stations with eight unknown coordinates. The elements of the X matrix thus consist of the dx and dy terms (called corrections). The value of K matrix is derived by subtracting computed quantities (distance and angles), based on the initial coordinate. So initial values denoted by $(\text{quantity})_{\circ}$.

$$X = \begin{bmatrix} d_{xb} \\ d_{yb} \\ d_{xc} \\ d_{yc} \\ d_{xd} \\ d_{yd} \\ d_{xe} \\ d_{ye} \end{bmatrix} \qquad K = \begin{bmatrix} D_{AB} - D_{AB\circ} \\ D_{BC} - D_{BC\circ} \\ D_{CD} - D_{CD\circ} \\ D_{DE} - D_{DE\circ} \\ D_{EF} - D_{EF\circ} \\ \bar{1} - \bar{1}\circ \\ \bar{2} - \bar{2}\circ \\ \bar{3} - \bar{3}\circ \\ \bar{4} - \bar{4}\circ \\ \bar{5} - \bar{5}\circ \\ \bar{6} - \bar{6}\circ \end{bmatrix}$$

Calculate the Jacobin matrix. The J matrix is formed using prototype equation for distance and angles. Since the units of the K matrix that relate to the angles are in seconds, the angle coefficients of the J matrix must multiplied by ° =206264.8 rad/sec

.

Distance observation equation:

$$\frac{x_{i\circ} - x_{j\circ}}{IJ_\circ} dx_i + \frac{y_{i\circ} - y_{j\circ}}{IJ_\circ} dy_i + \frac{x_{j\circ} - x_{i\circ}}{IJ_\circ} dx_j + \frac{y_{j\circ} - y_{i\circ}}{IJ_\circ} dy_j = k_{lij} + \neg_{lij} \quad \text{………………...……………….. (5.9)}$$

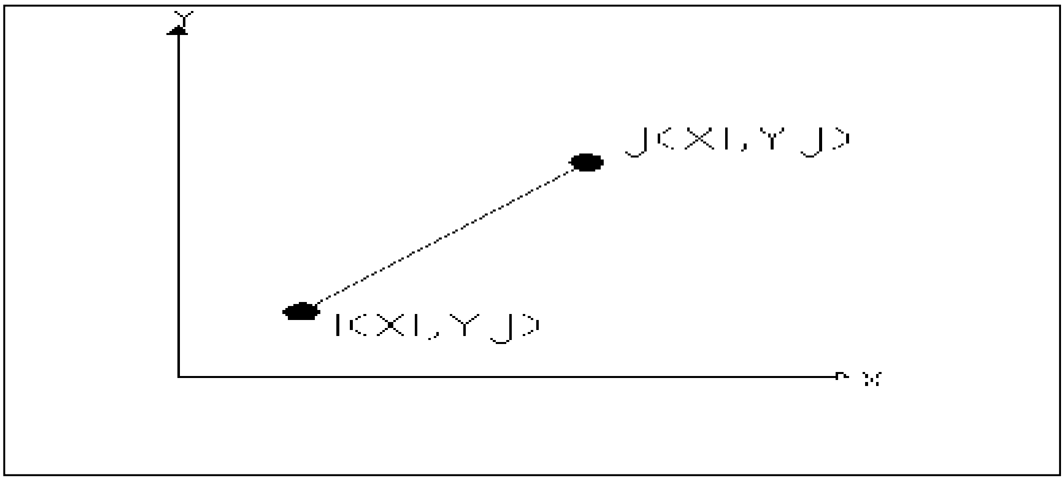


Figure (5.7): Measurement of distance

Angle observation prototype:

$$\frac{y_{i\circ}-y_{b\circ}}{(IB_{\circ})^2}dx_b + \frac{x_{b\circ}-x_{i\circ}}{(IB_{\circ})^2}dy_b + \left(\frac{y_{b\circ}-y_{i\circ}}{(IB_{\circ})^2} - \frac{y_{f\circ}-y_{i\circ}}{(IF_{\circ})^2}\right)dx_i +$$

$$\left(\frac{x_{i\circ}-x_{b\circ}}{(IB)^2} - \frac{X_{i\circ}-x_{f\circ}}{(IF_{\circ})^2}\right)dy_i + \frac{y_{f\circ}-y_{i\circ}}{(IF_{\circ})^2}dx_f + \frac{x_{i\circ}-x_{f\circ}}{(IF_{\circ})^2}dy_f = \ldots\ldots\ldots \quad (5.10)$$

$$k_{-bif} + \neg_{-bif}$$

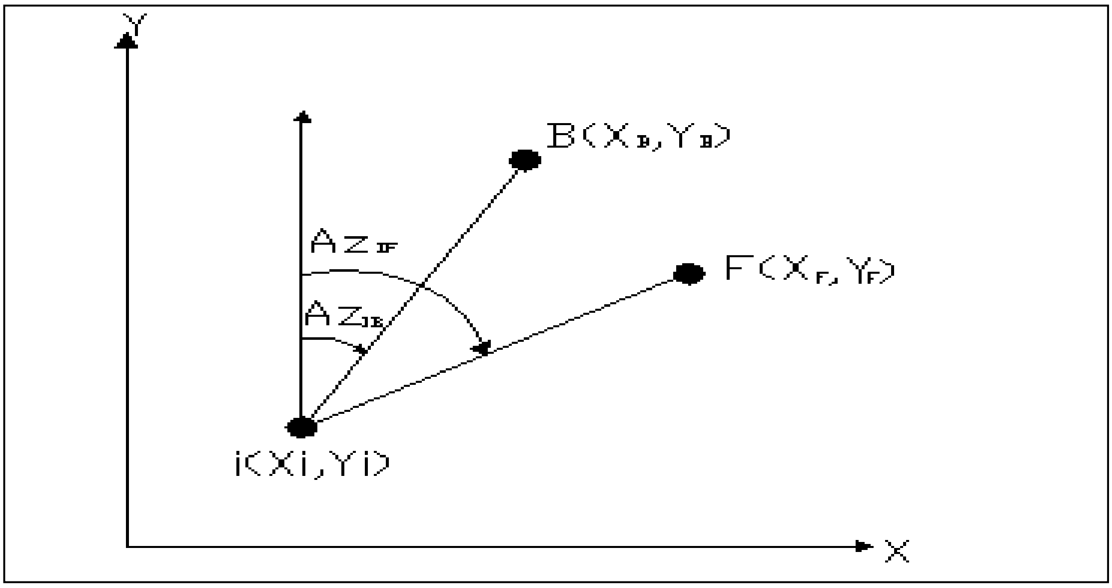


Figure (5.8): Relationship between an angle and two azimuths.

Formulate the W matrix. We are using the results from equations of error propagation to compute the values of weight matrix.

Distance: $w_{lij} = \dfrac{1}{\pm^{2}_{lij}}$ and angles: $w_{lij} = \dfrac{1}{\pm^{2-}_{bif}}$ ……………………………..…(5.11)

$$W=\begin{bmatrix}
\frac{1}{\pm^{2}_{AB}} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0\\
0 & \frac{1}{\pm^{2}_{BC}} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0\\
0 & 0 & \frac{1}{\pm^{2}_{CD}} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0\\
0 & 0 & 0 & \frac{1}{\pm^{2}_{DE}} & 0 & 0 & 0 & 0 & 0 & 0 & 0\\
0 & 0 & 0 & 0 & \frac{1}{\pm^{2}_{EF}} & 0 & 0 & 0 & 0 & 0 & 0\\
0 & 0 & 0 & 0 & 0 & \frac{1}{\pm^{2}{}_{-1}} & 0 & 0 & 0 & 0 & 0\\
0 & 0 & 0 & 0 & 0 & 0 & \frac{1}{\pm^{2}{}_{-2}} & 0 & 0 & 0 & 0\\
0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{1}{\pm^{2}{}_{-3}} & 0 & 0 & 0\\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{1}{\pm^{2}{}_{-4}} & 0 & 0\\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{1}{\pm^{2}{}_{-5}} & 0\\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \frac{1}{\pm^{2}{}_{-6}}
\end{bmatrix}$$

Solve the matrix system. This problem is iterative and need more than one iteration to reach minimum corrections.

J * X = K + V……………………………….……………………..……..…..(5.12)

J * X = K. …………………………………..…….…………..…………...(5.13)

$X = (J^{T} * W * J)^{-1}\ J^{T} *W*K.$ ……………..……………..…...…………….(5.14)

$X = (N)^{-1}\ J^{T} *W*K$

$X = (Q_{XX})^{-1}\ J^{T} *W*K.$ ………………………………………………………(5.15)

After final iteration adjusted coordinates are computed, also residuals, dusted distances, adjusted angles, and reference standard deviation is computed.

V = J * X – K ……………………………...………………………………(5.16)

$$S_{\circ} = \sqrt{\frac{V^{T} * W * V}{m - n}}$$ ……………….……………………..………………………(5.27)

m: number of observation.

n: number of unknowns.

Compute the standard deviations for the adjusted coordinates of traverse stations, using the appropriate elements from the Q_{XX} or (N^{-1}) matrix with the following equations.

$$\pm_{xp} = \pm s_{\circ} * \sqrt{\pm^{2}{}_{xxP}}$$ …………….…………………….....…..……………..…(5.18a)

$$\pm_{Yp} = \pm s_{\circ} * \sqrt{\pm^{2}{}_{yyP}}$$ …….……………………………………..………………(5.18b)

Where; P: point symbol.

$$Q_{XX} = \begin{bmatrix} \pm_{xxA} & \pm_{xyA} & & & & & & \\ \pm_{xy}A & \pm_{yy}A & & & & & & \\ & & \pm_{xxB} & \pm_{xyB} & & neglibilterms & & \\ & & \pm_{xyB} & \pm_{yyB} & & & & \\ & & & & \pm_{xxC} & \pm_{xyC} & & \\ neglibilterms & & & & \pm_{xxC} & \pm_{yyC} & & \\ & & & & & & \pm_{xxD} & \pm_{xyD} \\ & & & & & & \pm_{xyD} & \pm_{yyD} \end{bmatrix}$$

Table 5.9: Jacobean matrix for the plane traverses

0.001	0.001	0.000	0.000	0.000	0.000	0.000	0.000
-0.001	-0.001	0.001	0.000	0.000	0.000	0.000	0.000
0.001	0.000	-0.001	0.000	0.001	0.000	0.000	0.000
0.000	0.000	0.001	0.000	-0.002	0.000	0.001	0.000
0.000	0.000	0.000	0.000	0.001	0.000	-0.002	0.000
0.000	0.000	0.000	0.000	0.000	0.000	0.001	0.000
-0.778	0.628	0.000	0.000	0.000	0.000	0.000	0.000
-0.198	-0.980	0.198	0.980	0.000	0.000	0.000	0.000
0.118	0.993	-0.118	-0.993	0.118	0.993	0.000	0.000
0.000	0.000	0.000	0.000	-0.162	-0.987	0.162	0.987
0.000	0.000	0.000	0.000	0.000	0.000	0.340	-0.941

Table 5.10: K-matrix for the plane traverses

AB	-0.0000170626
BC	-0.0000170272
CD	-0.0000164546
DE	-0.0000165145
EF	-0.0000163685
¯1	-0.0000154879
¯2	0.4186759634
¯3	-0.0150626202
¯4	-0.2006364694
¯5	-0.3497001229
¯6	-0.1274234078

Table 5.11: Inverse matrix for the plane traverses

0.143	-0.116	0.139	-0.133	0.129	-0.213	0.113	-0.311
-0.116	0.094	-0.112	0.108	-0.104	0.171	-0.091	0.251
0.139	-0.112	0.164	0.010	0.153	-0.075	0.114	-0.314
-0.133	0.108	0.010	0.815	0.015	0.856	-0.086	0.237
0.129	-0.104	0.153	0.015	0.147	-0.036	0.105	-0.291
-0.213	0.171	-0.075	0.856	-0.036	1.186	-0.159	0.440
0.113	-0.091	0.114	-0.086	0.105	-0.159	0.090	-0.250
-0.311	0.251	-0.314	0.237	-0.291	0.440	-0.250	0.692

Table 5.12: Residuals for the plane traverses

AB	-0.418
BC	-0.084
CD	0.201
DE	0.35
EF	0.127
¯1	3.44
¯2	3.64
¯3	3.33
¯4	3.32
¯5	3.5
¯6	3.3

Table 5.13: corrections (x-matrix) for the plane traverses

POINT	X-CORRECTIONS	Y-CORRECTIONS
B	-0.0000108745	0.0000087561
C	-0.0000112606	0.0000069821
D	-0.0000152758	-0.0000263818
E	-0.0000075320	0.0000209279

Table 5.14: Final coordinates for plane traverses

POINT	X-CORRECTIONS	Y-CORRECTIONS
B	158817.650	106447.607
C	158591.110	105324.905
D	158404.853	103760.988
E	158230.494	102699.401

Table 5.15: Adjusted Distance for plane traverses

LINE	DISTANCE
AM	1021.639
AB	1136.599
BC	1145.330
CD	1574.969
DE	1075.810
EF	1605.322
FN	1357.078

Table 5.16: Adjusted angles for plane traverses

ANGLE	VALUE ° ' "
¯1	44 13 29.56
¯2	242 30 28.7
¯3	175 23 1.34
¯4	182 32 7.68
¯5	150 49 10.5
¯6	312 28 53.7

5.6.2 Computation of precise traverses at the grids plane

Procedure for precise traverse computation vary, depending on whether geodetic or a plane reference system is used. There are two basic corrections for precise traverse:

5.6.2.1 Correction for distance

Table 5.17: Zenith observations

FROM	TO	ZENITH ANGLE ° ' "	ELEVATION DIFFERENT aH	SLOPE DISTANCE (m)	REFLECTOR HEIGHT (m)	INSTRUMENT HEIGHT (m)
M	A	90 09 37.74	-2.871	1021.457	1.65	1.65
A	M	89 50 20.26	2.871	1021.457	1.65	1.65
A	B	91 45 45	-34.964	1136.722	1.65	1.65
B	A	88 14 18	34.964	1136.722	1.65	1.65
B	C	92 50 29.33	-56.845	1196.759	1.65	1.65

C	B	87 09 30	56.845	1196.759	1.65	1.65
C	D	88 47 22	33.285	1575.51	1.65	1.65
D	C	91 12 39	-33.285	1575.51	1.65	1.65
D	E	89 30 41	9.179	1076.160	1.65	1.65
E	D	90 29 19.67	-9.179	1076.160	1.65	1.65
E	F	91 41 46	-47.54	1606.158	1.65	1.65
F	E	88 18 46.06	47.54	1606.158	1.65	1.65
F	N	91 48 27.57	-42.825	1357.475	1.65	1.65
N	F	88 11 32.43	42.825	1357.475	1.65	1.65

Table 5.18: The elevation of the control point and the stations

STATIONS NO.	ELEVATIONS (m)
M (control point)	1022.27
A (control point)	1020.04
B	985.076
C	928.229
D	961.514
E	970.693
F (control point)	923.153
N (control point)	978.83

Reduction distance from slope distance to ellipsoid distance

Compute the scale factor, which related to Cassin projection.

Or S.F = grid distance from the coordinate /measured distance.

Compute the grid distance by multiplying the ellipsoidal distance by scale factor.

$G = (S.F)QL$

Where G is the grid distance at the projection plane $S.F$ is the scale factor.

Example 2: The scale factor between the control point (M, A) and (F, N).

- The scale factor between the control points M, A is:

Grid distance = $\sqrt{(X_2 - X_1)^2 + (Y_2 - Y_1)^2}$

$= \sqrt{(107256.16 - 107161.35)^2 + (158950.33 - 157933.10)^2}$

= 1021.638m

Ellipsoid chord distance (QL) = 1020.876516

Scale factor = $\frac{\text{Gd}}{\text{QL}} = \frac{1021.638}{1020.87516} = 1.000747241$

- The scale factor between the control point F, N is:

Grid distance = $\sqrt{(X_2 - X_1)^2 + (Y_2 - Y_1)^2}$

$= \sqrt{(101189.49 - 101711.63)^2 + (158775.68 - 157523.07)^2}$

= 1357.078m

Ellipsoid chord distance (QL) = 1356749669m

Scale factor = $\frac{\text{Gd}}{\text{QL}} = \frac{1357.078}{1356.749669} = 1.000241998$

Average scale factor = $\frac{1.000747241 + 1.000241998}{2} = 1.00049462$

Example 3: Reduction distance from Geode distance to Grid distance

FROM	TO	ZENITH ANGLE	SLOPE DISTANCE	ELEVATION
A	B	91 45 45	1136.722	1020.04 (A)
B	A	88 14 17.58	1136.722	984.46 (B)

From the equation (4.3)

$$\text{H.D} = \frac{\text{s.d}*sin¥}{sin(90 - c)}$$

$= 1136.722*\sin(88^{\circ}\ 13''\ 57.66') \big/ \sin(90+0^{\circ}\ 0''\ 18.41')$

$= 1136.181283\text{m}$

$c = (16.192 / \text{Km})(\text{s.d}(\text{Km})) = 0^{\circ}\ 0''18.41'$

From the equation (4.4)

$¥ = 180 - (¤ + 90 + c) = 88\ 13\ 57.66$

From the equation (4.5)

$$¤ = \frac{z_1 - z_2}{2} = 1\ 45\ 43.93$$

From the equation (4.6)

Chord distance = $QL = \frac{(R)(\overline{op})}{R+h_{o'}}$

$= (6371000)(1136.181283) \big/ (6371000+985.076)$

$= 1136.005635\text{m}$

$F = G / S$

$G = F * S = 1136.005635 * 1.00049462 = 1136.5688m$

Table 5.19: Reduction distance from slope distance to ellipsoid distance

FROM	TO	SLOPE DISTANCE (M)	HORIZONTAL DISTANCE (H.D) (m)	CHORD DISTANCE (QL)(m)	GRID DISTANCE (G.D)(m)
M	A	1021.475	1021.040	1020.876	1020.639
A	B	1136.722	1136.181	1136.006	1136.568
B	C	1146.759	1145.282	1145.108	1145.674
C	D	1575.510	1575.154	1574.925	1575.704
D	E	1076.160	1076.120	1075.958	1076.490
E	F	1606.158	1605.448	1605.216	1606.010
F	N	1357.617	1356.937	1356.750	1356.078

5.6.2.2 Reduction Azimuth to grid bearing

Example 4: calculate (t-T)" corrections for the first angle

STATION	E (Y)	N (X)
M	158950.33	107256.16
A	157933.10	107161.35
B	158817.312	106447.855

From the equation (4.8a)

$(t-T)''_{A,M} = (2\,a E_A + a E_M)(N_A - N_M)K$

$a E_A = 157933.10 - 500,000 = -342066.84$

$a E_M = 158950.33 – 500,000 = -341049.67$

$N_A - N_M = 107161.35 – 107256.16 = -94.81$

$(t-T)''_{A,M} = \left[2(-342066.84) + -341049.67)\right] (-94.81)\ 845*10^{-6}$

$(t-T)''_{A,M} = 0.082''$

From the equation (4.8b)

$(t\text{-}T)''_{A,B} = (2\,a\,E_A + a\,E_B)(N_A - N_B)\,K$

$a\,E_A = 157933.10 - 500{,}000 = -342066.84$

$a\,E_B = 158817.312 – 500{,}000 = -341182.688$

$N_A - N_B = 107161.35 – 106447.855 = 713.495$

$(t\text{-}T)''_{A,B} = \left[2(-342066.84) + -341182.688)\right] (713.495)\ 845*10^{-6}$

$(t\text{-}T)''_{A,B} = -0.592''$

From the equation (4.9) we find:

Corrected angle = observed angle - $(t\text{-}T)''_{A,\,M}$ - $(t\text{-}T)''_{A,\,B}$

$$= 44^{\circ}13'33'' - 0.082'' - (-0.592'') = 44^{\circ}13'33.51''$$

Table 5.20: (t-T)" corrections value, and the corrected angles.

BACK SIGHT	OCCUPIED	FORESIGHT	OBSERVED ANGLE ° ' "	(t-T)" CORRECTIONS "	CORRECTED ANGLE ° ' "
M	A		44 13 33	0.082	44 13 33.51
	A	B		-0.592	
A	B		242 30 32.34	0.618	242 30 31
	B	C		0.647	
B	C		157 23 04.67	0.971	157 23 5.05
	C	D		-1.354	
C	D		182 32 11	1.354	182 32 10.5
	D	E		-0.9197	
D	E		150 49 14	0.9198	150 49 14.3
	E	F		-1.307	
E	F		312 28 57	1.306	312 28 55.2
	F	N		0.452	

Table 5.21: Final coordinates for precise traverses

POINT	X-CORRECTIONS	Y-CORRECTIONS
B	158817.184	106447.977
C	158590.800	105326.048
D	158404.667	103763.214
E	158230.117	102700.478

Table 5.22: Adjusted Distance for precise traverses

LINE	DISTANCE (M)
AM	1021.639
AB	1136.004
BC	1144.541
CD	1573.879
DE	1076.975
EF	1606.463
FN	1357.078

Table 5.23: Adjusted angles for precise traverses

ANGLE	VALUE ° ' "
¯1	44 13 30.46
¯2	242 30 27.4
¯3	175 23 2.23
¯4	182 32 8.14
¯5	150 49 11.7
¯6	312 28 51.7

5.7 CHECK THE ORDER OF THE SURVEY

5.7.1 Evaluations the local accuracies for traverse (relative error ellipses)

Compute the elements (a_1, a_2, t) of relative error ellipse between two points or line. Construct all the possible pairs for the checkpoints and the available Geodetic Network Points (GNP). In our case the $\pm_{xs}, \pm_{ys}$ for the GNP are not available. Then assume $\pm_{xs}, \pm_{ys}$ for GNP are zeros, so we construct the pairs with the existing traverse points, using the following equations, so if we have tow points as 1, 2, then,

$$\begin{aligned} dx &= x_2 - x_1 \\ dy &= y_2 - y_1 \end{aligned} \quad \text{...} \quad (5.21)$$

$$J = \begin{bmatrix} -1 & 0 & 1 & 0 \\ 0 & -1 & 0 & 1 \end{bmatrix}$$

$$\Sigma_{1,2} = \begin{bmatrix} \pm^2_{X1} & \pm_{X1Y1} & \pm_{X1X2} & \pm_{X1Y2} \\ & \pm^2_{Y1} & \pm_{Y1X2} & \pm_{Y1Y2} \\ & & \pm^2_{X2} & \pm_{X2Y2} \\ symmetric & & & \pm^2_{Y2} \end{bmatrix}$$

$$J^T = \begin{bmatrix} -1 & 0 \\ 0 & -1 \\ 1 & 0 \\ 0 & 1 \end{bmatrix}$$

$$\Sigma_{D1,2} = J * \Sigma_{1,2} * J^T \quad \text{..} \quad (5.22)$$

According to error propagation law from which, relative covariance matrix for line 1-2 is:

$$\Sigma_{D1,2} = \begin{bmatrix} \pm^2_{dx} & \pm_{dxdy} \\ \pm_{dxdy} & \pm^2_{dy} \end{bmatrix}$$

$\pm^{2}_{dx} = \pm^{2}_{x1} - 2\pm_{x1x2} + \pm^{2}_{x2}$ ....(5.23a)

$\pm^{2}_{dy} = \pm^{2}_{y1} - 2\pm_{y1y2} + \pm^{2}_{y2}$ ....(5.23b)

$\pm_{dxdy} = \pm_{x1y1} - \pm_{x1y2} - \pm_{x2y1} + \pm_{x2y2}$ ....(5.23c)

Substitute the values from relative covariance matrix in the characteristic polynomial:

$a^{2} - (\pm^{2}_{dx} + \pm^{2}_{dx}) * a + (\pm^{2}_{dx} * \pm^{2}_{dy} - \pm^{2}_{dxdy}) = 0$ ....(5.24)

$a_{1} = value.$

$a_{2} = value.$

a_{1}, a_{2} Called eigenvalues of the covariance matrix,

Semi major axis $a_{1,2} = \sqrt{a_{1}} = \pm$

Semi minor axis $b_{1,2} = \sqrt{a_{2}} = \pm$

Tan (2t) = $2 \dfrac{\pm_{dxdy}}{\pm^{2}_{dy} - \pm^{2}_{dx}}$

t: orientation of the error ellipse.

Compute the circular standard deviation (circular error).

If $\pm_{\min} = b_{1,2}$ and $\pm_{\max} = a_{1,2}$ then if $\dfrac{\pm_{\min}}{\pm_{\max}}$ is between 0.1 and 0.6 then circular error $\pm_{c}$ = 0.5(a + b).

Then compute $\pm_{c}$ at a certain level of confidence (63.2%), then,

$\pm_{c}63.2\% = 1.414 * \pm_{c}$ (the value 1.1414 is from table (4.8)).

Then compute the relative precision a = $\dfrac{d}{\pm_{c}63.2\%}$

Consider the worst pair of points (the maximum value).

Determine the order and Class of the survey work according to table (4.4).

Example 5: Evaluate the local accuracies for traverse (relative error ellipses).

Calculation for line BC.

$$J = \begin{bmatrix} -1 & 0 & 1 & 0 \\ 0 & -1 & 0 & 1 \end{bmatrix}$$

$$\Sigma_{1,2} = \begin{bmatrix} 0.143 & -0.116 & 0.139 & -0.133 \\ -0.116 & 0.094 & -0.112 & 0.108 \\ 0.139 & -0.112 & 0.164 & 0.010 \\ -0.133 & 0.108 & 0.010 & 0.815 \end{bmatrix}$$

$$J^T = \begin{bmatrix} -1 & 0 \\ 0 & -1 \\ 1 & 0 \\ 0 & 1 \end{bmatrix}$$

$$J * \Sigma_{1,,2} * J^T = \Sigma_{Dbc} = \begin{bmatrix} -0.094 & 0.139 \\ 0.139 & 0.694 \end{bmatrix}$$

Then $\pm dx^2 = -0.094, \pm dy^2 = 0.694, \pm dx \pm dy = 0.139$

Use the characteristic polynomial.

$$a^2 - (\pm^2 dx + \pm^2 dy) * a + (\pm^2 dx * \pm^2 dy - \pm^2 dxdy) = 0$$

$$a^2 - (-0.094 + 0.694) * a + (-0.094 * 0.694 - 0.139) = 0$$

Then a_1, a_2 eigenvalues of the covariance matrix.

$a_1 = 0.718.$

$a_2 = 0.118.$

Semi major axis a= $\sqrt{a_1}$ = 0.847.

Semi minor axis b= $\sqrt{a_2}$ = 0.344.

Scaling to probability 63.2% by multiplying $\pm c$ with 1.414:

S = 1.414 * 0.847 = 1.198

Distance between the survey points = 1145.33.

Ratio d/s = 955.914.

Table 5.24: local accuracies for traverse (relative error ellipses).

LINE	**a = d/s**
BC	955.914
CD	1853
DE	751.2644

According to table above the maximum value (ratio) used to determine the order and class of the survey, according to table 4.4 this value corresponds to a third order, class 2.

5.7.2 Evaluations the network accuracies for traverse (standard error circle)

Consider the general case where the covariance matrix for the position of point (p) is given as:

$$\sum = \begin{bmatrix} \pm x^2 & \pm xy \\ \pm xy & \pm y^2 \end{bmatrix}$$

$$a^2 - (\pm^2 dx + \pm^2 dy) * a + (\pm^2 dx * \pm^2 dy - \pm^2 dxdy) = 0$$

Then a_1, a_2 eigenvalues of the covariance matrix.

Semi major axis a= $\sqrt{a_1}$

Semi minor axis b= $\sqrt{a_2}$

Compute ratio b/a to determine which equation is used to compute $\pm c$.

Scaling $\pm c$ to probability 63.2% by multiplying $\pm c$ with 1.414:

$\pm c_b = \pm c * 1.414$

Example 6: Evaluate the network accuracies for traverse (standard error circle). Calculation for point B.

According to equations (5.23a), (5.23b), and (5.23c) we find

$\pm xb^2 = -0.143, \pm yb^2 = 0.094, \pm x \pm yb = -0.116$

Using the characteristic polynomial.

$a^2 - (\pm^2 dx + \pm^2 dy) * a + (\pm^2 dx * \pm^2 dy - \pm^2 dxdy) = 0$

$a^2 - (-0.143 + 0.094) * a + (-0.143 * 0.094 - 0.116) = 0$

Then a_1, a_2 eigenvalues of the covariance matrix.

$a_1 = 0.237.$

$a_2 = 0.0000591$

Semi major axis a= $\sqrt{a_1}$ = 0.487.

Semi minor axis b= $\sqrt{a_2}$ = 0.007668.

Ratio b/a = 0.016

$\pm c$ = 0.5 (a + b) = 0.247.

Scaling $\pm c$ to probability 63.2% by multiplying $\pm c$ with 1.414:

S = 1.414 * 0.487 = 0.35.

Table 5.25: Network accuracies for traverse (standard error circle).

STATIONS	STANDARD ERROR CIRCLE
B	0.35
C	0.925
D	1.04
E	0.639

According to table above the maximum value used to determine network accuracy classification, referring to table 4.7 the network accuracy classification is 1 meter.

5.8 MAP ASSESSMENT

There is no matching between the control points and the block map points, as average positioning error, which is the arithmetic mean of all positioning errors between GCPs and block points before displacement correction.

5.8.1 Map assessment for plane traverses

Table 5.26: Positioning errors between the grounds check points (GCPs) a municipal topographic map points by plane traverses

	FIELD SURVEYING COORDINATES		**MAPS COORDINATES**			
P.NO	**X**	**Y**	**X**	**Y**	**X-DIFFERENCES**	**Y-DIFFERENCES**
1	106889.0737	158558.3087	106889.07	158559.5	0.0037	-1.1913
2	106888.3031	158572.4191	106888.3	158571.65	0.0031	0.7691
3	106967.8018	158585.3157	106967.93	158583.91	-0.1282	1.4057
4	106973.1168	158571.6938	106973.12	158571.64	-0.0032	0.0538
5	107016.8683	158580.4669	107016.5	158579.71	0.3683	0.7569
6	107020.033	158589.3559	107020.06	158589.39	-0.027	-0.0341
7	107070.4871	158594.73	107070.35	158594.61	0.1371	0.12
8	107087.565	158595.9359	107087.22	158595.83	0.345	0.1059
9	107076.7651	158608.752	107077.03	158609.35	-0.2649	-0.598
10	107082.1643	158586.5879	107082	158586.63	0.1643	-0.0421
11	107091.9861	157935.1086	107090.68	157936.23	1.3061	-1.1214
12	107156.2135	157984.2136	107155.2	157984.24	1.0135	-0.0264

13	106617.1637	158814.557	106615.64	158814.33	1.5237	0.227
14	106647.6653	158801.7527	106648.18	158802.14	-0.5147	-0.3873
15	106657.2875	158784.8335	106657.32	158783.99	-0.0325	0.8435
16	106651.1631	158770.5201	106649.58	158769.41	1.5831	1.1101
17	106509.0714	158794.2585	106508.85	158794.62	0.2214	-0.3615
18	106438.7693	158837.2339	106439	158837.01	-0.2307	0.2239
19	106467.0329	158851.4592	106466.85	158851.35	0.1829	0.1092
20	106319.3074	158860.329	106319.68	158859.68	-0.3726	0.649
21	105666.5184	158592.1245	105666.98	158591.28	-0.4616	0.8445
22	105483.0563	158579.3089	105482.35	158578.69	0.7063	0.6189
23	105464.2567	158566.1065	105464.04	158566.93	0.2167	-0.8235
24	105175.0719	158571.8172	105179.21	158572.55	-4.1381	-0.7328
25	105254.816	158569.1483	105254.37	158569.87	0.446	-0.7217
26	102804.1754	158288.9294	102805.56	158288.89	-1.3846	0.0394
27	102716.8766	158434.0285	102716.77	158431.9	0.1066	2.1285
28	102341.5641	158697.0425	102341.74	158697.32	-0.1759	-0.2775
29	102335.9383	158713.7664	102335.89	158714.84	0.0483	-1.0736
30	102200.884	158663.4115	102201.19	158662.37	-0.306	1.0415
31	101186.3123	158766.7798	101186.6	158765.51	-0.2877	1.2698
32	101820.2377	158848.6853	101820.86	158847.36	-0.6223	1.3253
33	101855.3947	158731.5292	101855.92	158730.72	-0.5253	0.8092

Some Statistical Analysis relates to positioning errors of table (5.26):

Number of data elements = (n) = 33

Minimum value = 0.043495

Maximum value = 4.202483

Standard deviation (S) = 0.786452

Mean = 0.906169

Median = 0.851535

Mode = 1.191306

5.8.1.1 Numerical results of block displacement correction:

$$MSE_{(X)} = \frac{\sum (V_X)^2}{n}$$
$$MSE_{(Y)} = \frac{\sum (V_Y)^2}{n} \quad \text{...} (5.22)$$

Where, MSE: is the Mean Square Error of positioning errors values.

n : Total number of points.

V: is residual in values of positioning errors, and can be calculated as:

$$V_X = X_{map} - X_{GCP}$$
$$V_Y = Y_{Map} - Y_{GCP} \quad \text{...} (5.23)$$

Then the total RMS error in X and Y directions are:

$$RMS_{(X)} = \sqrt{MSE_{(X)}}$$
$$RMS_{(Y)} = \sqrt{MSE_{(Y)}} \quad \text{...} (5.24)$$

Where, RMS: is the Root Mean Square error.

The total Root Mean Square Positional Error (RMSPE) can be determined by:

$$RMSPE = \sqrt{MSE_{(X)} + MSE_{(Y)}}$$

5.8.1.2 Testing for plane traverses of data above according to:

1. **National Map Accuracy Standards**

In order to make assessment to the Hebron municipal topographic map, a sample of (33) check points were surveyed and adopted in order to check them for positioning errors according to the ***National Map Accuracy Standards (*****NMAS)**, as over all the total population of all planimetric features of the map, and several statistical operations was made to show how the positioning errors between these check points and the coincided planimetric (house corners) points of the municipal topographic map were tends.

To apply the *National Map Standards of Accuracy* requirements of municipal topographic maps to accept it, It is required (90) percent of the principal planimetric features be plotted to within (1/30 *in*) of their true positions for map scale.

$$HP_{Error} = \left(\frac{1}{30}\right) * (2.54) * MapScale$$

Hebron municipal topographic map was compiled with scale (**1: 2500**), for that the acceptable or allowable positioning error for (90) percent of all planimetric features is:

$$HP_{Error} = \left(\frac{1}{30}\right) * (2.54) * 2500 = 211.667 \text{ cm} = \mathbf{2.1167m} \approx \mathbf{2.12m}.$$

In previous, it was described in table (5.26) that a table was established between ground check points-coordinates and coordinates of municipal topographic map points, differences in X, and Y coordinates was calculated, and the positioning errors "residuals" also calculated. With this table we can find the number of points that have positioning error more than the allowable (2.12 m) error as following:

In this case we have only two (2) points have positioning error more than allowable positioning error.

A simple check can be made to see that (90) % of all planimetric features in this sample are less than the allowable error as:

$$\left(\frac{2}{36}\right) * 100\ \% = 5.5\ \% \text{ is lower than } 10\ \% \quad \text{(accepted).}$$

2. **United State Geological Survey (USGS) Sstandards**

In order to make assessment to the Hebron municipal topographic map, a sample of (33) check points were surveyed and adopted in order to check them for positioning errors according to the ***(*USGS)**, as over all the total population of all planimetric features of the map, and several statistical operations was made to show how the positioning errors between these check points and the coincided planimetric (house corners) points of the municipal topographic map were tends.

To apply the United State Geological Survey (USGS) Sstandards, It is required (90) percent of the principal planimetric features be plotted to within (1/30 *in*) of their true positions for map scale.

$$HP_{Error} = \left(\frac{1}{30}\right) * (2.54) * MapScale$$

Hebron municipal topographic map was compiled with scale (**1: 2500**), for that the acceptable or allowable positioning error for (90) percent of all planimetric features is:

$$HP_{Error} = \left(\frac{1}{30}\right) * (2.54) * 2500 = 211.667 \text{ cm} = \mathbf{2.1167m} \approx \mathbf{2.12m}.$$

In previous, it was described in table (5.26) that a table was established between ground check points-coordinates and coordinates of municipal topographic map points, differences in X, and Y coordinates was calculated, and the positioning errors "residuals" also calculated. With this table we can find the number of points that have positioning error more than the allowable (2.12 m) error as following:

In our case we have only two (2) points have positioning error more than allowable positioning error.

A simple check can be made to see that (90) % of all planimetric features in this sample are less than the allowable error as:

$$\left(\frac{2}{36}\right) * 100\ \% = 5.5\ \% \text{ is lower than } 10\ \% \quad \text{(accepted).}$$

3. American Society for Photogrammetry and Remote Sensing (ASPRS, 1989)

In order to make assessment to the Hebron municipal topographic map, a sample of (33) check points were surveyed and adopted in order to check them for positioning errors according to the **(ASPRS, 1989)**, as over all the total population of all planimetric features of the map, and several statistical operations was made to show how the positioning errors between these check points and the coincided planimetric (house corners) points of the municipal topographic map were tends.

In table (5.26), Diff-X and Diff-Y considers as residuals denoted by (V) as:

V_X = Diff-X

V_Y = Diff-Y

The total *Root Mean Square* **(RMS)** error in X and Y–directions can be calculated as:

$$RMS_{(X)} = \sqrt{\frac{\sum (V_X)^2}{n}}$$

$$RMS_{(Y)} = \sqrt{\frac{\sum (V_Y)^2}{n}}$$

Where (n) is the number of points (n = 33).

For that:

$$RMS_{(X)} = \sqrt{\frac{29.37}{33}} = 0.94\text{m}$$

$$RMS_{(Y)} = \sqrt{\frac{22.7}{33}} = 0.83\text{m}$$

According to table (3.1) the value of RMSx = 0.94, the typical map scale must be 1/4000.

4. Federal Geographic Data Committee (FGDC, 1998)

In order to make assessment to the Hebron municipal topographic map, a sample of (33) check points were surveyed and adopted in order to check them for positioning errors according to the ***Federal Geographic Data Committee*** **(FGDC, 1998)**, as over all the total population of all planimetric features of the map, and several statistical operations was made to show how the positioning errors between these check points and the coincided planimetric (house corners) points of the municipal topographic map were tends.

In table (5.26), Diff-X and Diff-Y considers as residuals denoted by (V) as:

V_X = Diff-X

V_Y = Diff-Y

The total *Root Mean Square* **(RMS)** error in X and Y–directions can be calculated as:

$$RMS_{(X)} = \sqrt{\frac{\sum (V_X)^2}{n}}$$

$$RMS_{(Y)} = \sqrt{\frac{\sum (V_Y)^2}{n}}$$

Where (n) is the number of points (n = 33).

For that:

$$RMS_{(X)} = \sqrt{\frac{29.37}{33}} = 0.94\text{m} \quad RMS_{(Y)} = \sqrt{\frac{22.7}{33}} = 0.83\text{m}$$

$$RMSE_T = \sqrt{RMS_x^{\ 2} + RMS_y^2}$$

$$RMSE_T = \sqrt{0.94^2 + 0.83^2} = 1.33$$

Where the total Root Mean Square Positional Error (RMS_T) equal RMS_T = 1.33 m

t-distribution can be made to the sample of positioning errors that compares the relationship between the population variance and the variance of the sample set.

As (95%) is the confidence interval, which adopted, and number of reading is 33 equal (n-1=32).

$1.33 * 2.042$ = 2.71, Then the test is successesed at 95% confidence level

$5\% * 33 = 1.65 \approx 2$, Then 2 point allowable to exceed this value (2.71).

From table (5.26) there is one value greater than 2.71, so the map is with the specifications and standards.

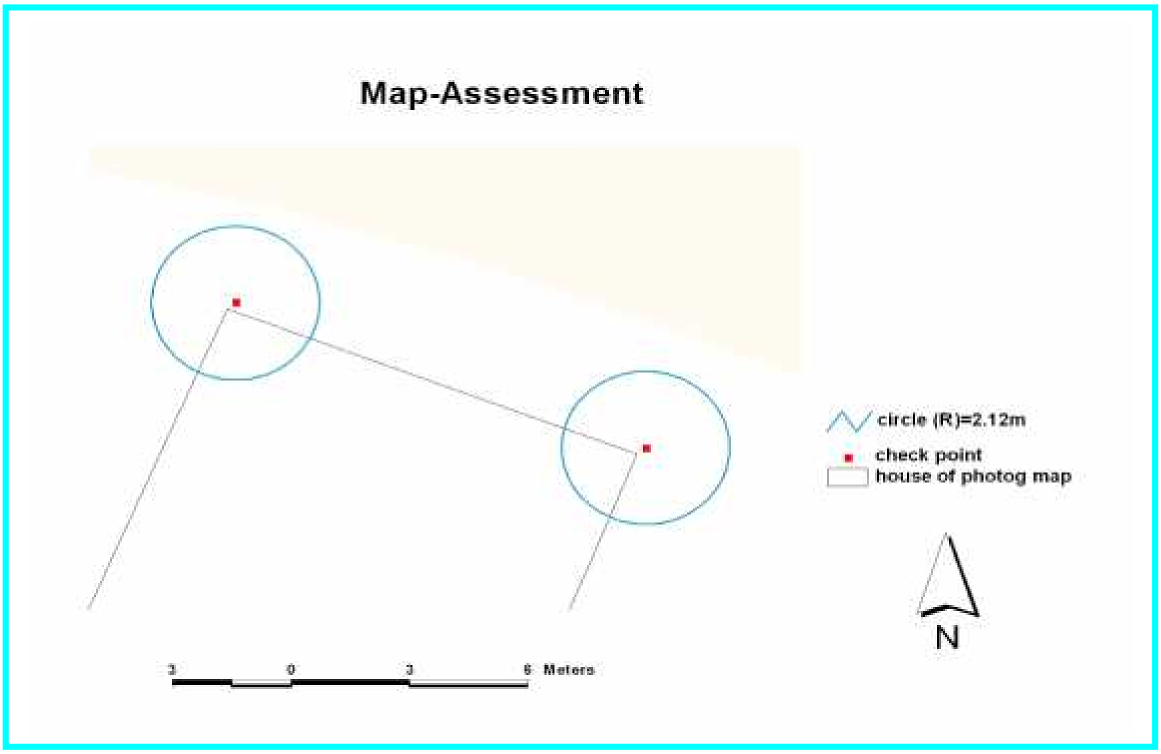


Figure (5.9): A sample of one house of municipal topographic map and three grounds checkpoints (accepted)

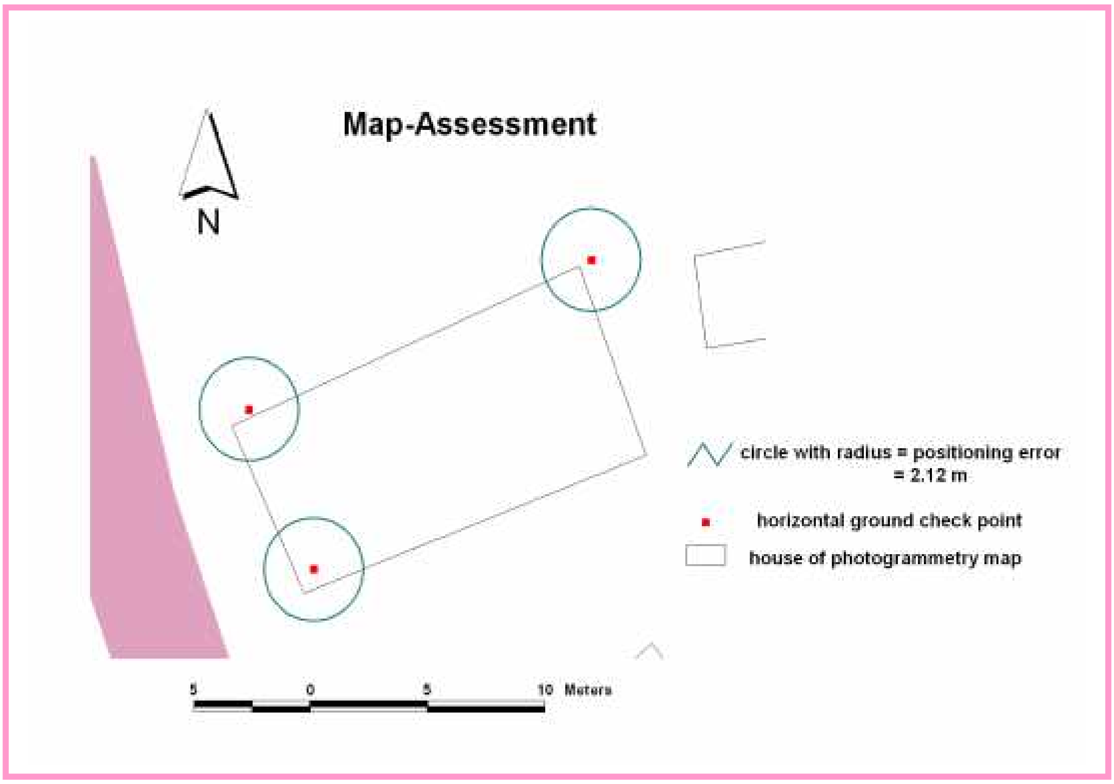


Figure (5.10): A sample of two corners of a house with two ground checkpoints (accepted)

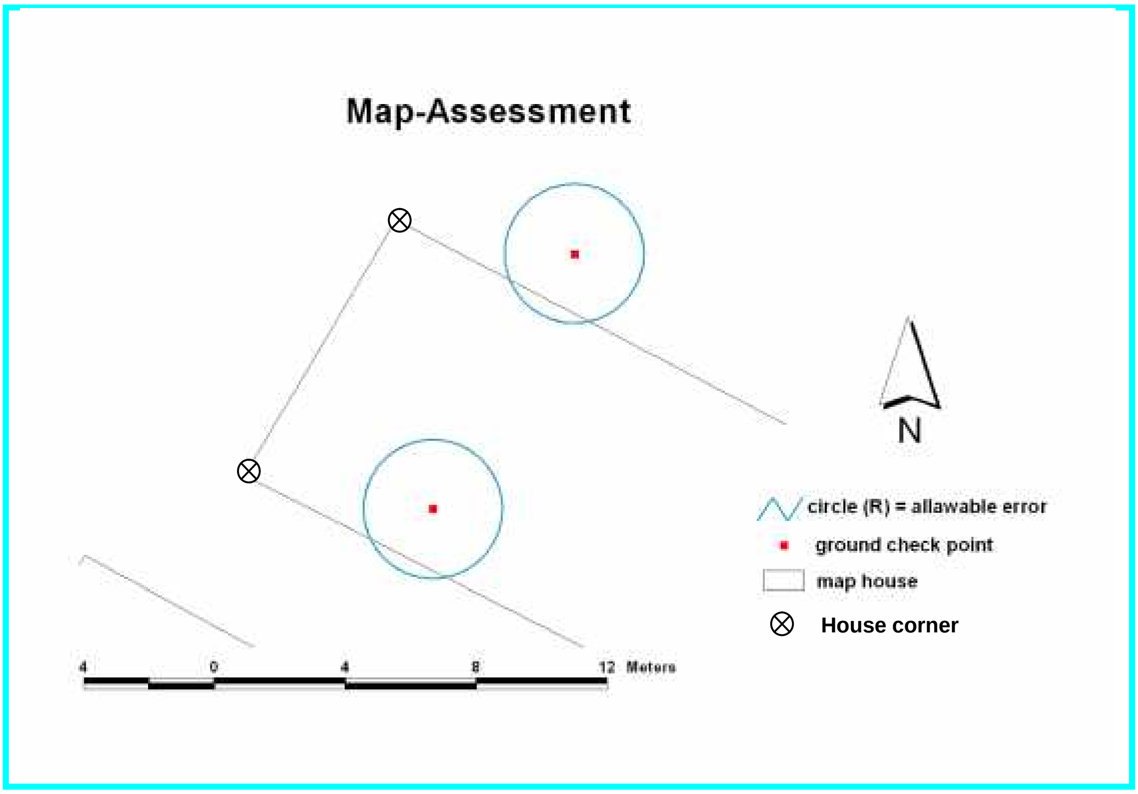


Figure (5.11): A sample of two corners of a house with two ground checkpoints (not accepted)

Figure (5.11) shows that the two house corners not included in the allowable positioning error circle, so it is considered that these planimetric features (two corners) failed to pass the test

5.8.2 Map assessment for precise traverses

Table 5.27: Positioning errors between the grounds check points (GCPs) a municipal topographic map points by precise traverses

	FELID SURVEYING COORDINATES		MAPS COORDINATES			
PO. NO	**X**	**Y**	**X**	**Y**	**Y-DIFFE RENT**	**X-DIFFERENT**
1	106889.07	158558.31	106889.07	158559.5	0	-0.19
2	106888.3	158572.42	106888.3	158571.65	0	0.28
3	106967.8	158585.32	106967.93	158583.91	-0.13	1.42
4	106973.12	158571.69	106973.12	158571.64	0	0.09
5	107018.56	158580.84	107016.5	158579.71	2.06	1.14
6	107020.03	158589.36	107020.06	158589.39	-0.03	-0.04
7	107071.64	158594.89	107070.35	158594.61	1.29	0.29
8	107087.57	158595.94	107087.22	158595.83	0.35	0.14
9	107076.77	158608.75	107077.03	158609.35	-0.26	-0.65
10	107082.16	158586.59	107082	158586.63	0.16	-0.01
11	107091.59	157935.12	107090.68	157936.23	0.91	-1.08
12	107156.21	157984.21	107155.2	157984.24	1.01	0.01
13	106617.53	158814.09	106615.64	158814.33	1.89	-0.21
14	106648.04	158801.29	106648.18	158802.14	-0.14	-0.81
15	106657.66	158784.37	106657.32	158783.99	0.34	0.37
16	106651.53	158770.05	106649.58	158769.41	1.95	0.65
17	106501.03	158796.99	106508.85	158794.62	1.45	2.37
18	106439.14	158836.77	106439	158837.01	0.14	-0.31
19	106467.37	158850.94	106466.85	158851.35	0.52	-0.44
20	105669.7	158593.93	106319.68	158859.68	0.12	0.25
21	105667.66	158591.82	105666.98	158591.28	0.068	0.51
22	105484.2	158578.99	105482.35	158578.69	1.85	0.27
23	105465.4	158565.8	105464.04	158566.93	1.36	-0.99

24	105176.21	158571.51	105179.21	158572.55	-3	-1.04
25	105255.96	158568.84	105254.37	158569.87	1.59	-1.05
26	102805.25	158288.55	102805.56	158288.89	-0.4	-0.34
27	102717.95	158433.65	102716.77	158431.9	1.18	1.75
28	102342.64	158696.66	102341.74	158697.32	0.9	-0.66
29	102337.01	158713.39	102335.89	158714.84	1.12	-1.45
30	102201.96	158663.03	102201.19	158662.37	0.77	0.66
31	101186.14	158766.84	101186.6	158765.51	-0.46	1.33
32	101820.07	158848.75	101820.86	158847.36	-0.79	1.39
33	101855.22	158731.59	101855.92	158730.72	-0.7	0.87

Some Statistical Analysis relates to positioning errors of table (5.27):

Number of data elements = (n) =33

Minimum value = 0.05

Maximum value = 3.175154

Standard deviation (S) = 1.169651

Mean = 1.02813429

Median = 1.116065

Mode = 0.19

5.8.2.1 Testing for precise traverses of data above according to:

1. **National Map Accuracy Standards**

To apply the *National Map Standards of Accuracy*, It is required (90) percent of the principal planimetric features be plotted to within (1/30 *in*) of their true positions for map scale.

$$HP_{Error} = \left(\frac{1}{30}\right) * (2.54) * MapScale$$

Hebron municipal topographic map was compiled with scale (**1: 2500**), for that the acceptable or allowable positioning error for (90) percent of all planimetric features is:

$$HP_{Error} = \left(\frac{1}{30}\right) * (2.54) * 2500$$ = 211.667 cm = **2.1167m ≈ 2.12m**.

In this case we have only three (3) points have positioning error more than allowable positioning error.

A simple check can be made to see that (90) % of all planimetric features in this sample are less than the allowable error as:

$\left(\frac{3}{36}\right)$ * 100 % = 9 % is lower than 10 % (accepted).

2. **United State Geological Survey (USGS) Sstandards**

To apply the ***(USGS)*** requirements of municipal topographic maps to accept it, It is required (90) percent of the principal planimetric features be plotted to within (1/30 *in*) of their true positions for map scale.

$$HP_{Error} = \left(\frac{1}{30}\right) * (2.54) * MapScale$$

Hebron municipal topographic map was compiled with scale (**1: 2500**), for that the acceptable or allowable positioning error for (90) percent of all planimetric features is:

$HP_{Error} = \left(\frac{1}{30}\right) * (2.54) * 2500$ = 211.667 cm = **2.1167m** $\approx$ **2.12m**.

In our case we have only three (3) points have positioning error more than allowable positioning error.

A simple check can be made to see that (90) % of all planimetric features in this sample are less than the allowable error as:

$\left(\frac{3}{36}\right)$ * 100 % = 9 % is lower than 10 % (accepted).

3. American Society for Photogrammetry and Remote Sensing (ASPRS, 1989)

In table (5.27), Diff-X and Diff-Y considers as residuals denoted by (V) as:

$V_X =$ Diff-X

$V_Y =$ Diff-Y

The total *Root Mean Square* **(RMS)** error in X and Y–directions can be calculated as:

$$RMS_{(X)} = \sqrt{\frac{\sum (V_X)^2}{n}}$$

$$RMS_{(Y)} = \sqrt{\frac{\sum (V_Y)^2}{n}}$$

Where (n) is the number of points (n = 33).

For that:

$$RMS_{(X)} = \sqrt{\frac{40.25}{33}} = 1.1\text{m}$$

$$RMS_{(Y)} = \sqrt{\frac{26.47}{33}} = 0.9\text{m}$$

According to table (3.1) the value of RMSx = 1.1, the typical map scale must be 1/4000.

4. Federal Geographic Data Committee (FGDC, 1998)

In table (5.27), Diff-X and Diff-Y considers as residuals denoted by (V) as:

V_X = Diff-X

V_Y = Diff-Y

The total *Root Mean Square* **(RMS)** error in X and Y–directions can be calculated as:

$$RMS_{(X)} = \sqrt{\frac{\sum (V_X)^2}{n}}$$

$$RMS_{(Y)} = \sqrt{\frac{\sum (V_Y)^2}{n}}$$

Where (n) is the number of points (n = 33).

For that:

$$RMS_{(X)} = \sqrt{\frac{40.25}{33}} = 1.1\text{m}$$

$$RMS_{(Y)} = \sqrt{\frac{26.47}{33}} = 0.9\text{m}$$

$$RMSE_T = \sqrt{RMS_x^{\ 2} + RMS_y^2}$$

$$RMSE_T = \sqrt{0.9^2 + 1.1^2} = 1.41$$

Where the total Root Mean Square Positional Error (RMS_T) equal RMS_T = 1.41m.

t-distribution can be made to the sample of positioning errors that compares the relationship between the population variance and the variance of the sample set.

As (95%) is the confidence interval, which adopted, and number of reading is 33 equal (n-1=32).

$1.41 * 2.042 = 2.9$, Then the test is successesed at 95% confidence level

$5\% * 33 = 1.65 \approx 2$, Then 2 point allowable to exceed this value (2.9).

From table (5.27) there is one value greater than 2.9, so the map meet the specifications and standards rrquirments.

Chapter six

6

CONCLUSION AND RECOMENDATIONS

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

1. Hebron municipal topographic map passed the assessment in this project. Accuracy assessment of current Hebron municipal topographic map showed that it's within the acceptable error according to the method used.

2. Four different agencies specification were used in assessment for Hebron municipal topographic map.

3. Traverse computations, started and ended with control points not free of errors. The biggest portion of the more probable trig that has this error is Al-Ahli Trig; because it's original position was changed, after adding changes to the hospital.

4. The difference of distance between the slope distance at the earth surface and the distance at a distance above the Geoide is approximately **0.20m.** And the difference of distance between the distance at the grid plane and the distance at the Geoide surface is approximately **0.20m.**

5. By compassion between plane traverse and precise traverse, plane traverse gives more accurate coordinates than precise traverse because of narrow range of the study area, which need uses plane traverse.

6. Error ellipses tests was used for evaluating the local accuracies for traverse (relative error ellipses), the result a third order, class 2 surveys, and for evaluating the network accuracies for traverse (standard error circle), the result network accuracy classification is 1 meter.

6.2 RECOMMENDATIONS

1. Map assessment with photogrammetry techniques

Updating maps by field surveying method has many disadvantages especially with large maps that cover large areas like:

Uneconomical work: costly and time consuming.

Hard work: in some areas and some conditions.

It is highly recommended to update maps with photogrammetric techniques when instrument is available

2. Map Assessment to the entire Hebron municipal topographic map:

In this project, the assessment of the municipal topographic map was only made on the planimetric features of the study area with respect to horizontal position, but a complete assessment must be done over the entire full map, i.e. to distribute ground check points over the entire total Hebron municipal topographic map, also a complete assessment must be done to the other layers of the map like contours layer "Z-values", with adopting the suitable specifications and standards.

3. Standards and Specifications "local Standards":

Some global standards like the *National Map Standards of Accuracy* for topographic maps assessment were used in the project. We recommended working on local Palestinian Standards.

4. Control points Assessment:

Errors may be exist in the control points in the study area that considered as a base of all the surveying works for local surveyors, there for, we highly recommended to make a precise assessment of these control points using suitable field survey techniques.

5. Adjustment the field surveying work:

In order to get ground control in a very good and precise quality, many field and office techniques must be done like, a total stations errors adjustment.

CHAPTER 6

CONTROL POINTS

6.1 GENERAL

By definition, a control survey consists of determining the horizontal and vertical or spatial positions of arbitrarily located points. Traditionally, horizontal and vertical controls have been established separately, but with the advent of GPS, which provides both horizontal and vertical control in the same operation, this separation is rapidly disappearing.

A geodetic control survey, takes into account the shape of the earth and generally is used for primary control networks of large extent and high precision, such as those surveys established for continents, states, and counties. The bulk of the geodetic surveys performed currently are done with GPS for the horizontal positions but geodetic leveling still is used for precise vertical control. By virtue of the characteristics of the system and the reduction process. differential GPS automatically yields a geodetic horizontal survey.

An engineering control survey provides the horizontal and vertical control for the design and construction of private and public works. Depending on the size and scope of the project, such a survey may be geodetic but often is simply a plane survey for horizontal control with precise or differential leveling for vertical control. Ideally, the engineering survey should originate and close on horizontal and vertical control points in the national or state geodetic network. Naturally, GPS surveying methods also are applicable to engineering surveys.

The distinguishing feature of a topographic survey is the determination of the location, both in plan and elevation, of selected ground points that are necessary for plotting contour lines and the planimetric location of features on the topographic map. A topographic survey consists of:

1. establishing, over the area to be mapped. a system of horizontal and vertical controls, which consists of key stations connected by measurements of high precision; and
2. locating the details, including selected ground points, by measurements of lower precision from the control stations.

Topographic surveys fall roughly into three classes, according to the map scale employed as follows:

1. *Large scale* 1:1200(1 in. to 100 ft) or larger
2. *Intermediate scale* 1:1200 to 1:12,000 (1 in. to 100 ft to 1 in. to 1000 ft)
3. *Small scale* 1:12,000 (1 in. to 1000 ft) or smaller

Because of the range in uses of topographic maps and variations in the nature of the areas mapped, topographic surveys vary widely in character.

Topographic surveys can be performed by aerial photogrammetric methods, ground survey methods, or some combination of these two procedures. The largest portion of almost all of the small- and intermediate-scale as well as some large-scale topographic mapping now is performed by photogrammetric methods. This photogrammetric operation includes establishing portions of the horizontal control in addition to compilation of the topographic map. However, ground survey methods still are applicable for large-scale topographic mapping of small areas and for field completion surveys, which usually are needed for photogrammetrically compiled topographic maps. The discussions in this chapter are directed primarily toward the various procedures for topographic surveys by ground survey methods.

6.2 PLANNING THE SURVEY

The choice of field methods for topographic surveying is governed by

1. The intended use of the map.
2. The area of the tract.
3. The map scale.
4. The contour interval.

1. *Intended use of the map.* Surveys for detailed maps should be made by more refined methods than surveys for maps of a general character. For example, the earthwork

estimates to be made from a topographic map by a landscape architect must be determined from a map that represents the ground surface much more accurately in both the horizontal and vertical dimensions than one to be used in estimating the storage capacity of a reservoir. Also, a survey for a bridge site should be more detailed and more accurate in the immediate vicinity of the river crossing than in areas remote there from.

2. *Area of the tract.* It is more difficult to maintain a desired precision in the relative location of points over a large area than over a small area. Control measurements for a large area should be more precise than those for a small area.

3. *Scale of the map.* It sometimes is considered that, if the errors in the field measurements are no greater than the errors in plotting, the former are unimportant. But, because these errors may not compensate each other, the errors in the field measurements should be considerably less than the errors in plotting at the given scale. The ratio between field errors and plotting errors should be perhaps one to three.
The ease with which precision may be increased in plotting, as compared with a corresponding increase in the precision of the field measurements, points to the desirability of reducing the total cost of a survey by giving proper attention to the excellence of the work of plotting points, interpolation, and interpretation in drawing the map.

4. *Contour interval.* The smaller the contour interval, the more refined should be the field methods.

6.3 ESTABLISHMENT OF CONTROL

Control consists of two parts:

1. *horizontal control*, in which the planimetric positions of specific control points are located.
2. *vertical control*, in which elevations are established on specified bench marks located throughout the area to be mapped. This control provides the skeleton, which later is clothed with the *details*, or locations of such objects as roads, houses, trees, streams, ground points of known elevation, and contours.

On surveys of wide extent, relatively few stations distributed over the tract are connected by more precise measurements forming the *primary control;* within this system, other control stations are located by less precise measurements, forming the *secondary control.* For small areas, only one control system is necessary, corresponding in precision to the secondary control used for large areas.

6.3.1 Horizontal Ground Control Points (HGCPs):

Horizontal ground control points as they are usually defined are points whose horizontal positions are known relative to a ground coordinate system, may be arbitrary or real coordinate system such as "Palestinian Coordinate System". Horizontal position of a point means that coordinates (X,Y) or (E,N) of point are known with respect to horizontal datum "origin".

These (HGCPs) wanted for digital map assessment must have the following properties:

1. Well distribute within the map.
2. Common in both block map and municipal topographic map.
3. Well defined in both, block map and municipal topographic map, and also in the field.
4. Must be located in good sites in the field to survey them easily.
5. They must be measured to a very high of accuracy as they will be used as a reference for map assessment.

We can use as examples of (HGCPs) as: intersection of roads, manhole covers, corners of buildings as we use in the project, fence corners, power or telephone poles...etc. We want to use them for (Purposes of HGCPs):

a. Block warping or registration from these points or digitized parcels "vector data" verification.
b. Municipal topographic map assessment.

In the case of topographic map assessment, the ground control points are usually called *check points*.

6.4 HORIZONTAL CONTROL

Horizontal control can be established by GPS survey, total station system traverse, aerial photogrammetric methods, ordinary traverse, or trilateration and triangulation. Frequently, a combination of certain of these methods is used.

Horizontal control determination by aerial photogrammetric methods is feasible and particularly applicable to small-scale mapping of large areas. Note that traditional photogrammetric control surveys require a basic framework of horizontal control points established by GPS or total station traverse. However, if a GPS receiver is used aboard the aircraft procuring the aerial photography, the number of ground control points can be substantially reduced, although as yet, not eliminated.

6.4.1 Select Horizontal Ground Control Points (HGCPs):

Horizontal ground control points as they are usually defined are points whose horizontal positions are known relative to a ground coordinate system, may be arbitrary or real coordinate system such as "Palestinian Coordinate System". Horizontal position of a point means that coordinates (X,Y) or (E,N) of point are known with respect to horizontal datum "origin".

These (HGCPs) wanted for digital map assessment must have the following properties:

6. Well distribute within the map.
7. Common in both block map and municipal topographic map.
8. Well defined in both, block map and municipal topographic map, and also in the field.
9. Must be located in good sites in the field to survey them easily.
10. They must be measured to a very high of accuracy as they will be used as a reference for map assessment.

We can use as examples of (HGCPs) as: intersection of roads, manhole covers, corners of buildings as we use in the project, fence corners, power or telephone poles...etc. We want to use them for (Purposes of HGCPs):

c. Block warping or registration from these points or digitized parcels "vector data" verification.
d. Municipal topographic map assessment.

In the case of topographic map assessment, the ground control points are usually called *check points*.

6.5 VERTICAL CONTROL

The purpose of vertical control is to establish bench marks at convenient intervals over the area to serve:

1. As points of departure and closure for operations of topographic parties when locating details.
2. As reference marks during subsequent construction work.

Vertical control usually is accomplished by direct differential leveling, but for small areas or in rough country the vertical control is frequently established by trigonometric leveling .

All elevations for topographic mapping should be tied to bench marks that are referred to the North American Vertical Datum of 1988.

Specifications for first-, second-, and third-order differential levels are given in (Table 4.2, and Table 4.3 in chapter 4). These specifications may be relaxed somewhat depending on map scale, character of the terrain to be mapped, the contour interval desired, and ultimate use of the survey. Table 6.1 gives the ranges of approximate

TABLE 15. 1Topographic survey vertical control specifications

Scale of map	Type of control	Length of circuit (Km)	Max error of closure (mm)
Intermediate	Primary	2-30	12-72* $\sqrt{Km}$
	secondary	2-8	24-120* $\sqrt{Km}$
Large	Primary	2-8	12-24* $\sqrt{Km}$
	secondary	1-5	12-24* $\sqrt{Km}$

closures applicable to intermediate- and large-scale topographic mapping surveys. The smaller error of closure for a given map and type of control is used for very flat regions, where a contour interval of *0.5* m (1 ft) or less is required, and on surveys that are to be used to determine gradients of streams or to establish the grades of proposed drainage or irrigation systems. The higher errors of closure apply to surveys in which no more exact use is made of the results other than to determine the elevations of ground points for contours having *0.5*-, 2-, and 3-rn or 2-, *5*-, and 10-ft intervals.

Revised accuracy standards, such as those currently being developed by the Federal Geographic Data Committee, may be adopted in the near future. New standards such as these, if adopted, would supersede the values given in Table 6.1. Consequently, a constant surveillance of the publications of the surveyor's professional organizations is required to stay abreast of developments in the continuing evolution of accuracy standards.

When an adequate number of points, having known elevations referred to the datum and a reliable mathematical function to model the geoid in the region, are available elevations by differential GPS survey may be used . Bear in mm that elevations so determined with current methods may have errors in the 3- to 5-cm range. Specifications and standards governing GPS elevations can be found in FGCC (1989). When accuracies required for elevations permit, GPS can provide elevations with substantial improvement in speed of acquisition and economy.

6.6 HORIZONTAL AND VERTICAL CONTROL BY THREE-DIMENSIONAL TRAVERSE

A three-dimensional, total-station system traverse can be used for establishing control for intermediate- and large-scale topographic mapping jobs. Care should be exercised in the trigonometric leveling aspect, with changes in elevations being determined in both directions for each traverse line and corrections for earth curvature and refraction applied for all long lines.

Differential GPS surveys automatically provide the dimension. Such surveys are satisfactory for horizontal and vertical control establishment. for topographic surveys, assuming that the qualifications with respect to elevations, as detailed in the previous section, are present.

1. Scale of map
2. Type of control

6.7 UNITED STATES NATIONAL MAP ACCURACY STANDARDS

With a view to the at most economy and expedition in producing maps which fulfill not only the broad needs for standard or principal maps, but also the reasonable particular needs of individual agencies, standards of accuracy for published maps are defined as follows:

1. *Horizontal accuracy***.** For maps on publication scales larger than 1:20,000, not more than 10 percent of the points tested shall be in error by more than 1/30 inch, measured on the publication scale; for maps on publication scales of 1:20,000 or smaller, 1/50 inch. These limits of accuracy shall apply in all cases to positions of well-defined points only. Well-defined points are those that are easily visible or recoverable on the ground, such as the following: monuments or markers, such as bench marks, property boundary monuments; intersections of roads, railroads, etc.; corners of large buildings or structures (or center points of small buildings); etc. In

general what is well defined will be determined by what is plottable on the scale of the map within 1/100 inch. Thus while the intersection of two road or property lines meeting at right angles would come within a sensible interpretation, identification of the intersection of such lines meeting at an acute angle would obviously not be practicable within 1/100 inch. Similarly, features not identifiable upon the ground within close limits are not to be considered as test points within the limits quoted, even though their positions may be scaled closely upon the map. In this class would come timber lines, soil boundaries, etc.

2. *Vertical accuracy*, as applied to contour maps on all publication scales, shall be such that not more than 10 percent of the elevations tested shall be in error more than one-half the contour interval. In checking elevations taken from the map, the apparent vertical error may be decreased by assuming a horizontal displacement within the permissible horizontal error for a map of that scale.

3. *The accuracy of any map may be tested* by comparing the positions of points whose locations or elevations are shown upon it with corresponding positions as determined by surveys of a higher accuracy. Tests shall be made by the producing agency, which shall also determine which of its maps are to be tested, and the extent of the testing.

4. *Published maps meeting these accuracy requirements* shall note this fact on their legends, as follows: "This map complies with National Map accuracy Standards."

5. *Published maps whose errors exceed those foretasted* shall omit from their legends all mention of standard accuracy.

6. *When a published map is a considerable enlargement* of a map drawing (manuscript) or of a published map, that fact shall be stated in the legend. For

example, "This map is an enlargement of a 1:20,000-scale map drawing," or "This map is an enlargement of a 1:24,000-scale published map."

To facilitate ready interchange and use of basic information for map construction among all Federal mapmaking agencies, manuscript maps and published maps, wherever economically feasible and consistent with the uses to which the map is to be put, shall conform to latitude and longitude boundaries, being 15 minutes of latitude and longitude, or 7.5 minutes, or 3-3/4 minutes in size.

6.8 LOCATION OF DETAILS

In the following sections, the horizontal and vertical controls are assumed to have been established and the field party is concerned only with the location of details.

The adequacy with which the resulting map sheet meets the purposes of the survey depends largely on the task of locating the details. Therefore, the topographer should be informed as to the uses of the map so that proper emphasis is placed on each part of the work.

The instruments currently most used for the location of details are the total station system and GPS equipment. The engineer’s optical reading theodolite and engineer’s level with level rod or staid rod, although still used on certain types of topographic surveys, rapidly are being displaced by the previously mentioned electronic systems.

The principal procedure for acquiring topographic detail in the field, using current equipment (e.g., total station system, GPS, or theodolite staid), is the controlling point method. Other classical procedures, using theodolite, tape. rod, and the like, are the cross-profile, checkerboard, and trace-contour methods. With the prevalence of total station systems and GPS equipment in current surveying operations, much less emphasis now is placed on the checkerboard and trace contour methods for topographic surveying. Therefore. The major focus is on use of the controlling point method for locating topographic details.

6.9 PRECISION

The precision required in locating such definite objects as buildings, bridges, curbs, inlets, and boundary lines should be consistent with the precision of plotting, which may be assumed to be a map distance of about *0.5* mm or 1/50 in. Such less definite objects as shorelines, streams, and the edges of wooded areas are located with a precision corresponding to a map distance of perhaps 0.9 to 1.3 mm or 1/30 to 1/50 in. For use in maps of the same relative precision, more located points are required for a given area on large-scale surveys than on intermediate surveys; hence, the location of details is relatively more important on large-scale surveys.

6.10 Contours

The accuracy with which contour lines represent the terrain depends on

1. The accuracy and precision of the observations.
2. The number of observations.

3. The distribution of the points located.

Although ground points are definite, contour lines must necessarily be generalized to some extent. The error of field measurement in a plan should be consistent with the error in elevation, which in general should not exceed one-fifth of the horizontal distance between contours. The error in elevation should not exceed one-fifth of the vertical distance between contours. The purpose of a topographic survey will be better served by locating a greater number of points with less precision. within reasonable limits, than by locating fewer points with greater precision. Therefore. if for a given survey the contour Interval is 5 ft. a better map will be secured by locating with respect to each instrument station perhaps 50 points whose standard deviation in elevation is 1 ft than by locating 25 points whose standard deviation is only 0.5 ft. Similarly, if the contour interval is 2 m, it is better to have 50 points with a standard deviation of 0.4 m than 25 points with a standard deviation of 0.2 m.

A general principle that should serve as a guide in the selection of ground points may be noted. As an example, let it be supposed that a given survey is to provide a map that shall be accurate to the extent that if a number of well-distributed points is chosen at random on it, the average difference between the map elevations and ground elevations of identical points shall not exceed one-half of a contour interval. Under this require-ment, an attempt is made in the field to choose ground points such that a straight line between any two adjacent points in no case will pass above or below the ground by more than one contour interval. Therefore, if the ground points were taken only at *a, b. c, d,* and *e,* as shown, the resulting map would indicate the straight slopes *cd* and *de;* the consequent errors in elevation of *inn* and *op* on the profile amount to two contour intervals and show that additional readings should have been taken at the points *n* and *o*. The corresponding displacement of the contours on the map is shown by dashed and full lines.

6.11 Angles

The precision needed in the field measurements of angles to details may be readily deter-mined by relating it to the required precision of corresponding vertical and horizontal

distances. For a sight at a distance of 300 m (1000 ft), a permissible error of 0.09 m (0.3 ft) ~ in elevation corresponds to a permissible error of 01' in the vertical angle; likewise, a permissible error of 0.09 m (0.3 ft) in azimuth (measured along the arc from the point.

sighted) corresponds to a permissible error of 01' in the horizontal angle. Values for other lengths of sight or degrees of precision are obtained in a similar manner; therefore, if it is desired to locate a point to the nearest 2 ft in azimuth (or elevation) and if the length of the sight is 500 ft. the corresponding permissible error in the angle is 2/500 = 0.004 rad = 14'.

COMPUTATION OF PRECISE TRAVERSES AT THE GRIDS PLANE

Procedure for precise traverse computation vary, depending on whether geodetic or a plane reference system is used. There are two basic corrections for precise traverse:

1. Correction for distance

- Reduction of slope distance at the ground to horizontal distance (chord) at elevation above the ellipsoid.

  $$\overline{op} = \frac{\bar{o}p\sin¥}{\sin(90+c)}$$

  Where $\overline{op}$ horizontal distance at elevation above the ellipsoid, $\bar{o}p$ slope distance.

  $$¥ = 180^\circ - (¤ + 90^\circ + c)$$

  $$¤ = \frac{z_p + z_{o'}}{2}$$

  $c = (16.192'' / km)(\bar{o}p_{km})$ Defined curvature.

  Arc to chord correction

  $$p\bar{o} - s = \frac{k^2 s^3}{24R^2}$$

- Reduction of the horizontal distance to horizontal distance at the ellipsoid (ellipsoidal distance).

  $$QL = \frac{(R)(\overline{op})}{R + h_{o'}}$$

  QL Ellipsoid chord distance, R radius of the earth ellipsoid, $h_{o'}$ elevation of o' above the ellipsoid.

Arc to chord correction

B – QL = $(QL)^3 / (24R^2)$

- Compute the scale factor, which related to Cassin projection.

  Or S.F = grid distance from the coordinate /measured distance.
- Compute the grid distance by multiplying the ellipsoidal distance by scale factor.

  $G = (S.F)QL$

  Where G is the grid distance at the projection plane $S.F$ is the scale factor.

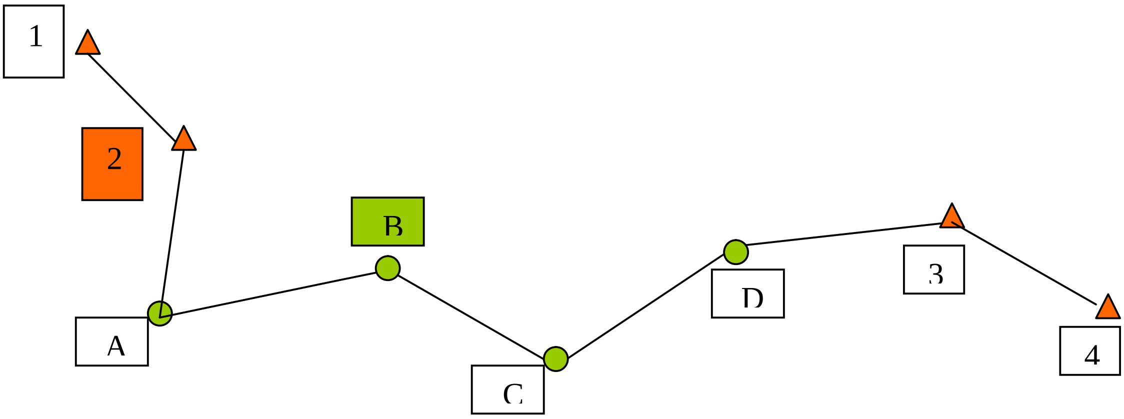
2. **Correction for angle**

- Geodetic azimuth can be transformed to grid bearing by the application of (t-T) correction.
- $(t\text{-}T)''AB = (2\Delta E_A + \Delta E_B)(N_{A-N_B})$ K
- $(t\text{-}T)''AC = (2\Delta E_B + \Delta E_A)(N_{B-N_A})$ K
- $= \bar{} + (t\text{-}T)''AB - (t\text{-}T)''AC$
- Where
- $(t\text{-}T)''$ AB = correction for line AB at station A.
- $(t\text{-}T)''$ AC = correction for line AC at station A.
- $=$ Computed angle from the grid coordinate.
- $\bar{} =$ Observed angle.
- $\Delta E =$ National grid easting – easting of the central meridian.
- N = national grid northing.
- K= $845*10^{-6}$

- Easting of the central meridian = 500000

Computing and analyzing link traverse misclosure errors

A link traverse begins in one station and ends on a different one. Normally, they are used to establish the positions of intermediate stations, as in A through D of the figure 1. The coordinates of the end points, stations 1, 2, 3, and 4 of the figure, are known. Angular and linear misclosures are also computed for these types of traverse, and the resulting values used as the basis for the accepting or rejecting the measurements.



ANGULAR MISCLOSURE:

IN LINKE TRAVERSE, ANGUKAR MISCLOSURE IS FOUND BY COMPUTING INITIAL AZIMUTHS FOR EACH COURSE, AND THEN SUBSTRACTING THE FINAL COMPUTED AZIMUTH FROM ITS GIVEN COUNTERPART. THE INITIAL AZIMUTHS AND THEIR ESTIMATED ERRORS ARE COMPUTED USING EQUATIONS **()** AND **()** AND THEIR TABLE IS SHOWN.

The difference between the azimuths computed foe course 34 = $\pm\pm_{differnce1}$.

Using equation **(5.1),** the estimated standard error in the difference is $\sqrt{\pm^2_{34copmuted} + \pm^2_{34actual}} = \pm\pm_{34}$, if the $\pm\pm_{differnce1}$ is less than or equal $\pm\pm_{34}$, thus there is no reason to assume that the angles contain blunders.

Linear misclosure:

$$A = \begin{bmatrix} \cos(Az_{1A}) & -D\sin(Az_{1A}) & 0 & 0 & 0 & 0 \\ \sin(Az_{1A}) & D\cos(Az_{1A}) & 0 & 0 & 0 & 0 \\ 0 & 0 & \cos(Az_{AB}) & -D\sin(Az_{AB}) & 0 & 0 \\ 0 & 0 & \sin(Az_{AB}) & D\cos(Az_{AB}) & 0 & 0 \\ . & . & . & . & . & . \\ . & . & . & . & . & . \\ . & . & . & . & . & . \\ 0 & 0 & 0 & 0 & \cos(Az_{34}) & -D\sin(Az_{34}) \\ 0 & 0 & 0 & 0 & \sin(Az_{34}) & D\cos(Az_{34}) \end{bmatrix}$$

$$\Sigma = \begin{bmatrix} \pm^2_{D1A} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \left(\frac{\pm_{Az1A}}{\circ}\right)^2 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & \pm^2_{DAB} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & \left(\frac{\pm_{AzAB}}{\circ}\right)^2 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & \pm^2_{DBC} & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & \left(\frac{\pm_{AzBC}}{\circ}\right)^2 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & \pm^2_{DCD} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & \left(\frac{\pm_{AzCD}}{\circ}\right)^2 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \pm^2_{DD3} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \left(\frac{\pm_{AzD3}}{\circ}\right)^2 \end{bmatrix}$$

TABLE: Distance observations

From	TO	Distance (meter)	S (meter)
1	2	Measured	Computed
2	A	Measured	Computed
A	B	Measured	Computed
B	C	Measured	Computed
C	D	Measured	Computed
D	3	Measured	Computed
3	4	Measured	Computed

TABLE: Angle observations

Back sight	Occupied	Foresight	Angle	S
2	1	A	Measured	Computed
1	A	B	Measured	Computed
A	B	C	Measured	Computed
B	C	D	Measured	Computed
C	D	3	Measured	Computed
D	3	4	Measured	Computed

TABLE: Control stations

Station	X (ft)	Y (ft)
1	Known	Known
2	Known	Known
3	Known	Known
4	Known	Known

TABLE: Azimuth observations

From	To	Azimuth	S
1	2	Computed	Computed
3	4	Computed	Computed

Table: Jacobian matrix

Note: the values of angles are in radians.

Table: inverse matrix

INVERSE							
0.143	-0.116	0.139	-0.133	0.129	-0.213	0.113	-0.311
-0.116	0.094	-0.112	0.108	-0.104	0.171	-0.091	0.251
0.139	-0.112	0.164	0.010	0.153	-0.075	0.114	-0.314
-0.133	0.108	0.010	0.815	0.015	0.856	-0.086	0.237
0.129	-0.104	0.153	0.015	0.147	-0.036	0.105	-0.291
-0.213	0.171	-0.075	0.856	-0.036	1.186	-0.159	0.440
0.113	-0.091	0.114	-0.086	0.105	-0.159	0.090	-0.250
-0.311	0.251	-0.314	0.237	-0.291	0.440	-0.250	0.692

Table5.21: Adjusted Distance

LINE	DISTANCE (M)
AM	1021.639
AB	1136.004
BC	1144.541
CD	1573.879
DE	1076.975
EF	1606.463
FN	1357.078

Table5.22: Adjusted angles

ANGLE	**VALUE** ° ' "
¯1	44 13 30.46
¯2	242 30 27.4
¯3	175 23 2.23
¯4	182 32 8.14
¯5	150 49 11.7
¯6	312 28 51.7

Table 5.13: corrections (x-matrix)

POINT	X-CORRECTIONS	Y-CORRECTIONS
B	-0.0000871210	0.0001081685
C	-0.0000730427	0.0000069821
D	0.0000557927	0.0001269529
E	-0.0002250664	0.0000810505

Table 5.14: Final coordinates for plane traverses

POINT	X	Y
B	158817.184	106447.977
C	158590.800	105326.048
D	158404.667	103763.214
E	158230.4943262010	102699.4005989320

Table 5.12: Residual

AB	-0.418
BC	-0.084
CD	0.201
DE	0.35
EF	0.127
¯1	3.44
¯2	3.64
¯3	3.33
¯4	3.32
¯5	3.5
¯6	3.3

Table 5.10: K-matrix

AB	-0.0000153269
BC	-0.0000152741
CD	-0.0000145036
DE	-0.0000145007
EF	-0.0000141276
¯1	-0.0000133045
¯2	0.5601415970
¯3	-0.0666088479
¯4	-0.2684114930
¯5	1.5115363177
¯6	2.0403606772

Tests for Accuracy of Maps and Map Data

- Maps can be tested for accuracy by several techniques.
- The ASPRS (1989) accuracy standard recommend that the conventional rectangular topographic maps be field checked with a minimum of **20** checkpoint , with **20%** of these point , located in each quadrant of the map sheet , are spaced at interval of at least **10%** of the sheet diagonal.
- In general, the spatial distribution of checkpoints should not be specified of large – scale , special purpose engineering maps. For these maps, checkpoints are to be concentrated in critical area containing structures and drainage facilities , with less dense concentration of the point in area where no construction is to occur.
- **Check surveys**
    - Check surveys should conducted according to the Federal Geodetic Control Subcommittee (FGCS) standard for vertical and horizontal net work (FGCS 1984;1994).
    - Levels to establish vertical control are classified as first, second , third order , depending on the equipment and procedures employed .Table 5.2 shows the classification and general specifications for first ,second ,and third order vertical control.

Table 5.2

Classification and general specification for vertical control (abstracted from FGCS ,Standard and Specification for Geodetic Control Network ,1984)

Order	**First**		**Second**		**Third**
Class	I	II	I	II	__
Net work geometry					
Bench mark spacing not to exceed (Km)	3	3	3	3	3
Average B.M spacing not to exceed (Km)	1.6	1.6	1.6	1.6	3.0
Distance between net work control points not to exceed (Km)	300	100	50	50	25
1nstrumement					
Leveling instrument					
Minimum repeatability of the line of sight	0.25”	0.25”	0.50”	0.50”	1.00”
Leveling rod construction	IDS	IDS	IDS or ISS	ISS	Wood metal
Instrument and rod resolution (combined) least count (mm)	0.1	0.1	0.5-1.0	1.0	1.0
Calibration procedure					
Leveling instrument					
Mix collimation error, single line of sight ,mm/m	0.05	0.05	0.05	0.05	0.01
Reversible compensator ,mean of two value mm/m	0.02	0.02	0.02	0.02	0.04
Time interval between compensator check not to exceed (day)reversible compensator	7	7	7	7	7
Other types (day)	1	1	1	1	1
Mix angular difference between two line of sight ,compensator	40”	40”	40”	40”	60”
Leveling rod					
Mix scale calibration	N	N	N	M	M
Time interval between scale calibration ,year	1	1	—	—	—
Leveling rod verticality to within	10’	10’	10’	10’	10’
Field procedure					
Minimal observation method	Micro(cm)	Micro(cm)	Micro(cm)	3-wire	Center

Section running	SRDS,DR or SP	SRDS,DR or SP	SRDS,DR or SP	SRDS, or DR	SRDS, or DR
Minimal observation method	Micro(M	Micro(M	Micro(M) or 3-wire	3-wire	Center wire
Different between forward and backward sights not to exceed					
Per setup (m)	2	5	5	10	10
Per section (m)	4	10	10	10	10
Max sight length (m)	50	60	60	70	90
Min ground clearance ,line of sight (m)	0.5	0.5	0.5	0.5	0.5
Even number setups when using rod without detailed calibration	Yes	Yes	Yes	Yes	—
Determine temperature gradient for vertical range of line of sight at each setup	Yes	Yes	Yes	—	—
Max section misclosure (mm)	$3\sqrt{D}$	$4\sqrt{D}$	$6\sqrt{D}$	$8\sqrt{D}$	$12\sqrt{D}$
Max loop misclosure (mm)	$4\sqrt{D}$	$5\sqrt{D}$	$6\sqrt{D}$	$8\sqrt{D}$	$12\sqrt{D}$
Single – run methods					
Reverse direction of single run every half	Yes	Yes	Yes	—	—
Nonreversible compensator levels					
Off-level/relevel between observing high and low rod scales	Yes	Yes	Yes	—	—
3-wire method					
Reading check (different between top and bottom intervals)for one setup not to exceed (tenths of rod units)	—	—	2	2	2
Read rod 1 first in alternate setup method	—	—	Yes	Yes	Yes
Double –scale rods					
Low-high scale elevation different for one setup not to exceed (mm) with reversible compensator	0.4	1.00	1.00	2.00	2.00
Other type of level					
Half-centimeter rods	0.25	0.30	0.60	0.70	1.30
Full-centimeter rods	0.30	0.30	0.60	0.70	1.30

- The specification given in table 5.2 were prepared by the (FGCC) now known as (FGCS).
- Standard of accuracy for leveling
- The FGCS (Bossler ,1984) also sets forth vertical network standard based on an elevation different accuracy.
  - Designated by b ,where $b = d/¢$

    In which, $d =$ approximate horizontal distance between control point position ,in Km.

    ¢ = estimated standard deviation of elevation different ,in mm, between control points propagated from a least square a adjustment.

    The unit of b are in mm/Km
  - Elevation different accuracy values for b are given for the various order of leveling in table 5.3

Table 5.3 Elevation accuracy standards

Classification	Max elevation different accuracy , b
First order ,class I	0.5
First order ,class II	0.7
Second order ,class I	1.0
Second order ,class II	1.3
Third order	2.0

Example 5.14

To calculate the standard devotions in the elevation difference.

line	Stand. Div. ,s ,mm	Length , d , Km	$b := \frac{s}{\sqrt{d}}$
1	37.9	24.1	7.72
2	41.9	19.3	9.53
3	48.2	45.1	7.18
4	$9*10^{-7}$	41.8	$1.4*10^{-7}$
5	40.5	27.4	7.74
6	30.9	17.7	7.34
7	30.9	20.9	6.76

The maximum value s/d = 9.53 for line 2 with the specified allowable amounts in table 5.3 shows that these levels do not qualify even for 3^{rd} order accuracy.

Standards of Accuracy for control surveys:

- The accuracy required for hori3ontal control depends on the type of surveys and ultimate use of the control points.
- The geodetic control surveys are prepared by the federal Geodetic control subcommittee (FGCS) in the USA, And reviewed toy the American society if civil Engineers, the American congress on Surveying and Mapping the American Geophysical union.

- As published in the 1984 FGCS Report (Bossler, 1984, these standards provided for three order of accuracy (first, second, and third), the latter of which are subdivided into class I and II.

- First-order or primary hori3ontal control provides the principal framework for the national control network. It is used for crystal movement studies in areas if seismic and tectonic activity, for testing defense and scientific equipment, for studying the performance of space vehicles, for engineering projects of high precision and extending over long distances, and for surveys used in metropolitan expansion.

- Second-order, class I or secondary control consists of networks between first-order arcs and detailed surveys where land values are high. Surveys of this call include the basic framework for densification for control-secondary hori3ontal control strengthens the entire network and is adjusted as part of the national network.

- Second-order, class II surveys are used to establish control for inland waterways, the interstate highway system, and extensive land subdivision and construction this class contributes to and is published as part of the national network.

- Third-order, class I and class II or supplementary surveys are used to establish control for local improvement and development. Topographic and hydrographic surveys, or other such projects for which they provide sufficient accuracy. Third-order control may or may not be adjusted to the national network. The surveyor engineer should know that third order, class I surveys constitute the lowest order permissible for specifying points on the state coordinate system.

- A specific order and class indicates that control points has a particular relationship with respect to all other points in the network. These relation- ship is expressed as mlative accuracy or ratio of the relative positional error to the distance between the separation of the points-control calcifications with corresponding relative accuracies or distance accuracy standards are shown in table 9.5.

Distance accuracy standards

Classification	minimum distance accuracy
First order	1: 100,000
Second order, class I	1: 50,000
Second order , class I I	1: 20,000
Third order , class I	1: 10.000
Third order , class I I	1 : 5,000

- The relative or distance accuracy 1: a, is determined from a least-squares adjustment using:

$$a = d / s$$

- where a is the inverse of the distance accuracy ratio. S is the propagated standard deviation of the distance between survey points obtained from the least – squares adjustment and d is the distance between survey points.

Theoretically, the distance accuracy is with respect to all pairs of points, but in practice only a sample of pairs is competed. The worst distance accuracy is used as the provisional accuracy.

consider the following equations to illustrate the process:

$$\sum zz=\begin{bmatrix} \pm^{2}{}_{XB} & \pm_{XBYB} & \pm_{XBXC} & \pm_{XBYC} \\ & \pm^{2}{}_{YB} & \pm_{YBXC} & \pm_{YBYC} \\ & & \pm^{2}{}_{XC} & \pm_{XCYC} \\ Symmetric & & & \pm^{2}{}_{YC} \end{bmatrix}$$

classify this survey according to the 1984 standards.

when we are interested in the relative accuracy between two points, 1 ad 2, in a horizontal network,

Then ,the coordinate differences are:

$dx = x_2 - x_1$ (1.1)

$dy = y_2 - y_1$

$$\Sigma dd = J * \Sigma_{1,2} * J^T$$

$$d = \begin{bmatrix} d_x \\ d_y \end{bmatrix} = \begin{bmatrix} -1 & 0 & 1 & 0 \\ 0 & -1 & 0 & 1 \end{bmatrix} * \begin{bmatrix} x_1 \\ y_1 \\ x_2 \\ y_2 \end{bmatrix} = JP1,2orJX$$

$$\Sigma_{dd} = \begin{bmatrix} -1 & 0 & 1 & 0 \\ 0 & -1 & 0 & 1 \end{bmatrix} * \begin{bmatrix} \pm^{2}{}_{x1} & \pm_{x1y1} & \pm_{x1x2} & \pm_{x1y2} \\ & \pm^{2} y1 & \pm_{y1x1} & \pm_{x2y2} \\ & & \pm^{2} x2 & \pm_{x2y2} \\ symmetric & & & \pm^{2}{}_{y2} \end{bmatrix} * \begin{bmatrix} \pm^{2} dx & \pm dxdy \\ \pm dydx & \pm^{2} dy \end{bmatrix}$$

$$\lambda^2 - (\sigma^2 x_{bc} + \sigma^2 y_{bc}) \cdot \lambda + (\sigma^2 x_{bc} \sigma^2 y_{bc} - \sigma^2 x_{bc} y_{bc}) = 0 \qquad (1.3)$$

$$\tan 2\bar{} = 2\sigma_{xy} / \sigma^2{}_{y} - \sigma^2{}_{xx} \qquad (1.4)$$

Now introducing the variance, $\sigma^2 dx, \sigma^2 dy$, and covariance, σ_{dxdy}, in Equation(1.3) and equation (1.4) the elements of relative error ellipse can be computed.

- In our example ,we have;

$$\sigma^2 xbc = \sigma \quad dx = \sigma^2 xb - 2\sigma xbxc + \sigma^2 xc$$

$$\sigma^2 ybc = \sigma^2 dy = \sigma \quad yb - 2\sigma ybyc + \sigma \quad yc$$

$$\sigma xbcybc = \sigma dxdy = \sigma xbyb - \sigma xbyc - \sigma xcyb + \sigma xcyc$$

$$\Sigma Dbc = \begin{bmatrix} \sigma^2 dx & \sigma dxdy \\ \sigma dydx & \sigma^2 dy \end{bmatrix}$$

$$\lambda^2 - (\sigma^2 xbc + \sigma^2 ybc) \cdot \lambda + (\sigma^2 xbc \sigma^2 ybc - \sigma^2 xbcybc) = 0$$

(1.3)

From the characteristic polynomial, using Equation (1.3)

From which.,

= value $\lambda 1$

= value $\lambda 2$

$\lambda 1 and \lambda 2$ are the semimajor and semiminor axis a and b of the standard relative error ellipse (with probability of 0.394) for the line bc so that

$a_{bc} = \sqrt{\lambda 1}$

$b_{bc} = \sqrt{\lambda 2}$

- In the ratio s/d from the standards, s is the standard deviations, so it is necessary to scale abc to a probability of 0.632 (see Miknail Gracieg 1981,p, 230) using the constant 1.414 to give abc = (1.414)(0.014) = 0.020 cm.

- These fore, the ratio d/s = 855 /0.020 = 42.7 so. Accordant to table g.5, this value corresponds to a second orders ,class II survey.

Proposed revised standards

- The 1984 standards currently are being reviewed and revised by the FGCS and the federal geographic data committee (FGDC); draft reports were released in December 1994 (FGCS,1994) and January 1997 (FGDC,1997).
- The emphasis in these documents is to propose use of statistically based standard, consisting of a number with stated probability, for the network and local surveys.
- The horizontal network accuracy of a control point is number corresponding to the radius of an absolute error circle ,with confidence level of 95percent relative to the geodetic datum.
- For the case ,the datum is defined by the nearest continuously operating reference station (CORS) in the national spatial reference system (NSRS) ,supported by the national geodetic (NGS).
- Local accuracy of a control point is shown by number that represents the relative error circle ,with a95 percent confidence level ,of that point with respect to known control points connected by the local survey.
- The local accuracy reported is the average of all the individual values of local accuracy along the lines involved in connecting the point to the local survey .
- Horizontal accuracy standards (there are also used for ellipsoid and optometric heights) are shown in table where classification is by groups of 1-millimeter, 2-millimeter, 5-millimeter, 5meter,and 10-meter accuracies.

Accuracy standards for horizontal position, ellipsoid height, and optometric

height (modified courtesy of FGDC,1997)

Accuracy classification	95% confidence, m (less than or equal to)
1 millimeter	0.001
2 millimeter	0.002
5 millimeter	0.005

1 centimeter	0.01
2 centimeter	0.02
5 centimeter	0.05
1 decimeter	0.1
2 decimeter	0.2
5 decimeter	0.5
1 meter	1.0
2 meter	2.0
5 meter	5.0
10 meter	10.0

To illustrate the value of these standard ,let us first discuss the concept of the error ellipse:

ERROR ELLIPSE:

1. the variance and standard deviation are measures of precision of the one-dimensional case of an angle or a distance .
2. in the 2_dimentional problems, such as the horizontal position of a point, error ellipses may be established around the point to designate precision regions of different probabilities.
3. the orientation of the ellipse relative to the x,y axes system fig( ) depends on the correlation between x and y. if they are uncorrelated, the ellipse axes will be parallel to x and y . if the two coordinates are equal precision, or $\pm x = \pm y$, the ellipse becomes a circle.
4. consider the general case where the covariance matrix for the position of point p is given as :

$$\Sigma = \begin{bmatrix} \pm^2 x & \pm xy \\ \pm xy & \pm^2 y \end{bmatrix}$$

5. the semimajor and semiminor axes of the corresponding ellipse are computed in the following manner :

- compute the tangle by:

$$\tan(2t) = 2\pm_{xy} / \pm^{2}_{y} - \pm^{2}_{xx} \qquad (1)$$

- the quadrant of 2t is determined according to the following signs:

Algebraic sign

	quadrant	numerator (sin2t)	dominator (cos2t)
0	1	+	+
180	2	+	-
270	3	-	-
360	4	-	+

$$\pm^{2}u = \pm^{2}x \sin^{2}(t) + 2\pm xy \cos(t)\sin(t) + \pm^{2}y \cos^{2}(t) \qquad (2)$$

$$\pm^{2}v = \pm^{2}x \cos^{2}(t) - 2\pm xy \cos(t)\sin(t) + \pm^{2}y \sin^{2}(t) \qquad (3)$$

the semimajor axis = $\pm u$ = a = $\sqrt{\pm^{2}u}$

the semiminor axis = $\pm v$ = b = $\sqrt{\pm^{2}v}$

6. the probability of falling on or inside the standard error ellipse is 0.394.
7. in manner similar to constructing intervals with given probabilities for the one-dimensional case, different size ellipses may be established, each with given probability. it should be obvious that the larger the size of the error ellipse, the larger is the probability.
8. using the standard ellipse as abase, table () gives the scale multiplier k to enlarge the ellipse and the corresponding probability.
9. as an example, for an ellipse with axes a=2.4476$\pm u$ and b= b=2.447$\pm v$, where $\pm u$ and $\pm v$ are the semimajor axes and semiminor axes, respectively, of the standard ellipse, the probability that the point fall inside the ellipse is 0.95.

table ( )

point estimate probability values

2_dimensions		3_dimensions	
p	k	p	k
0.394	1.000	0.199	1.000
0.500	1.177	0.500	1.538
0.632	1.414	0.900	2.500
0.900	2.146	0.950	2.700
0.950	2.447	0.99	3.368
0.990	3.035	____	____

10. in surveying, one frequently is interested in the relative accuracy between two points 1 and 2, in a horizontal network. Then, the coordinate difference are :

$dx = x_2 - x_1$

$dy = y_2 - y_1$

$$\Sigma 1,2 = \begin{bmatrix} \pm^2 x1 & \pm x1y1 & \pm x1x2 & \pm x1y2 \\ & \pm^2 y1 & \pm y1x1 & \pm x2y2 \\ & & \pm^2 x2 & \pm x2y2 \\ symmetric & & & \pm y2 \end{bmatrix} = JP1,2orJX$$

then,

$$d = \begin{bmatrix} dx \\ dy \end{bmatrix} = \begin{bmatrix} -1 & 0 & 1 & 0 \\ 0 & -1 & 0 & 1 \end{bmatrix} * \begin{bmatrix} x1 \\ y1 \\ x2 \\ y2 \end{bmatrix} = JP1,2orJX$$

$\Sigma dd = J * \Sigma 1,2 * J^T$ according to error propagation law

from which,

$$\pm^2 dx = \pm^2 x1 - 2\pm x1x2 + \pm^2 x2$$

$$\pm^2 dy = \pm^2 y1 - 2\pm y1y2 + \pm^2 y2$$

$$\pm dxdy = \pm x1y1 - \pm 2\pm x1y2 - \pm x2y1 + \pm x2y2$$

the variance, $\pm^2 dx, \pm^2 dy$ and covariance, $\pm dxdy$ of the line 1_2 have to be substituted into equations 1,2,and 3 to calculate the elements of relative error ellipse.

11. in three-dimensional case, where the horizontal position as well as the elevation of the points is involved, the precision region becomes an ellipsoid. Table( ) gives the corresponding multipliers.
12. the concepts of error ellipse and error ellipsoids are quite useful in establishing confidence regions about points determined by surveying techniques. These regions are measures of the ratability of the positional determination of such points. They could also be specified in advance as means of establishing specifications.
13. although both absolute error ellipses (for points) and relative error ellipses (for lines) are used to evaluate adjustment quality, it is frequently more convenient to replace the two-dimensional representation by one-dimensional single quantity (similar to ±). In this case, a circular probability distribution is substituted for the elliptical probability distribution.
14. consequently, a single circular standard deviation $\pm c$ is calculated from the two semi axes of the relative magnitudes of these axes.

Let $\Sigma xx = \begin{bmatrix} \pm^2 x & \pm xy \\ \pm xy & \pm^2 y \end{bmatrix}$ represent the covariance matrix for the x,y coordinate of a point. Then, $\pm^2 u = a$ and $\pm v = b$ are the semimajor and semiminor axis of the error ellipse, respectively.

The ratio $\pm v / \pm u$ =a/b determines the relationship used to calculate $\pm c$.

- When $\pm v / \pm u$ is between 1.0 and 0.6, then.

$$\pm c = 0.5222 \pm v + 0.4778 \pm u$$

- A good approximation that yields as lightly larger $\pm c$ (i.e. on the safe side) is given by :

$$\pm c = 0.5(\pm v + \pm u) = 0.5(a + b) \qquad \text{(c)}$$

which may be extended to the limit of $\pm \min / \pm \max = \pm v / \pm u \leq 0.2$

- Instead of equation (c) we can use :

$$\pm \bar{c} = \left[0.5(\pm^2 c + \pm^2 y)\right]^{1/2} \qquad \text{(d)}$$

this equation is applied only when $\pm v/\pm u$ is between 1.0 and 0.8, in which case it yields essentially the same value of $\pm c$ as in equation c.

- As the ratio of $\pm v/\pm u$ decreases, $\pm c$ from equation (d) gets progressively larger than that from equation (c) with maximum increase of about 20% at $\pm v/\pm u = 0.2$.
- Of course, the probability associated with standard error circle is the same as for the standard error ellipse, 0.394. the multipliers given in table( ) also still apply for a circular errors of different probabilities.

Table B-1: FIELD SURVY OBSERVATION TABLE

						Zenith Angle (B.S ° ' "		Zenith Angle (F.S) ° ' "		Horizontal Angle ° ' "			
Back sight	**Occupied**	**Fore sight**	**Slope Distance**	**Elevation Difference**	**Horizontal Distance**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Instrument Height**	**Reflector Height**
	ST 101												
ST 100			1021.475	2.871	1021.471	90 09 40	269 50 20			44 13 32	44 13 34	1.65	1.65
						90 09 38	269 50 22			44 13 34	44 13 36	1.65	1.65
						90 09 42	269 50 18			44 13 30	44 13 32	1.65	1.65
		ST 102	1136.722	-34.964	1136.181			91 45 44	268 14 14				
								91 45 46	268 14 16				
								91 45 42	268 14 12				
		1	682.946	-37.306	681.924	93 08 04				28 51 28		1.65	
		2	696.172	-37.011	695.186	93 02 00				28 27 06		1.65	
		3	681.407	-38.300	680.328	93 13 28				21 51 12		1.65	
		4	666.863	-.38.340	665.756	93 17 52				21 44 54		1.65	
		5	664.534	-40.456	663.294	93 00 04				17 54 22		1.65	
		6	672.515	-40.400	671.299	93 26 46				17 20 38		1.65	
		7	669.156	-41.924	667.840	93 35 40				13 02 40		1.65	
		8	668.268	-42.203	666.930	93 37 24				11 40 30		1.65	
		9	682.205	-41.713	680.926	93 30 30				12 27 38		1.65	
		10	659.629	-42.322	658.768	93 40 54				12 14 04		1.65	
		11	69.289	-3.649	69.793	93 01 06				93 39 58		1.65	
		12	51.500	-3.637	51.371	94 03 00				11 03 48		1.65	

Table B-4: FIELD SURVY OBSERVATION TABLE

						Zenith Angle (B.S) ° ' "		Zenith Angle (F.S) ° ' "		Horizontal Angle ° ' "			
Back sight	**Occupied**	**Fore sight**	**Slope Distance**	**Elevation Difference**	**Horizontal Distance**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Instrument Height**	**Reflector Height**
	ST 104												
ST 103			1575.51	-33.285	1575.170	91 12 38	268 47 22			182 32 10	182 32 10	1.65	1.65
						91 12 40	268 47 24			182 32 10	182 32 16	1.65	1.65
						91 12 42	268 47 20			182 32 14	182 32 06	1.65	1.65
		ST 105	1076.198	9.179	1076.160			89 30 40	270 29 18				
								89 30 42	270 29 20				
								89 30 38	270 29 16				

Table B-3: FIELD SURVY OBSERVATION TABLE

						Zenith Angle (B.S) ° ' "		Zenith Angle (F.S) ° ' "		Horizontal Angle ° ' "			
Back sight	**Occupied**	**Fore sight**	**Slope Distance**	**Elevation Difference**	**Horizontal Distance**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Instrument Height**	**Reflector Height**
	ST 103												
ST 102			1169.759	56.845	1145.345	87 09 32	272 50 28			175 23 06	175 23 02	1.65	1.65
						87 09 30	272 50 30			175 23 00	175 23 04	1.65	1.65
						87 09 28	272 50 32			175 23 08	175 23 08	1.65	1.65
		ST 104	1575.51	33.285	1575.170			88 47 22	271 12 38				
								88 47 24	271 12 40				
								88 47 26	271 12 36				
		1	344.022	-15.637	343.668	92 36 22				349 06 50		1.65	
		2	341.969	-15.543	341.615	92 36 24				349 45 44		1.65	
		3	325.081	-16.086	324.682	92 50 16				349 37 28		1.65	
		4	191.093	-18.049	190.219	95 25 12				345 31 38		1.65	
		5	190.139	-12.240	189.745	93 41 30				349 13 38		1.65	
		6	159.680	-18.614	158.591	96 41 30				344 14 20		1.65	
		7	142.816	-18.678	141.577	97 33 08				338 25 12		1.65	
		8	86.301	-18.569	84.250	102 25 30				329 30 10		1.65	
		9	96.212	-21.834	93.017	103 07 04				181 46 16		1.65	
		10	152.694	-22.210	151.070	98 21 50				195 55 44		1.65	
		11	219.119	-23.373	217.869	96 02 24				169 49 12		1.65	
		12	118.814	-21.884	116.381	100 36 52				166 33 46		1.65	
		13	55.122	-21.413	50.793	112 51 34				169 21 28			
		14	76.454	-21.225	73.449	106 07 06				185 54 22			

Table B-2: FIELD SURVY OBSERVATION TABLE

						Zenith Angle (B.S)		Zenith Angle (F.S)		Horizontal Angle			
	Occupied	**Fore sight**	**Slope Distance**	**Elevation Difference**	**Horizontal Distance**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Instrumen t Height**	**Reflector Height**
	ST102												
ST101			1136.722	34.964	1136.181	88 14 18	271 45 42			242 30 34	242 30 32	1.65	1.65
						88 14 20	271 45 44			242 30 28	242 30 36	1.65	1.65
						88 14 16	271 45 40			242 30 30	242 30 34	1.65	1.65
		ST103	1196.759	-56.845	1145.345			92 50 28	267 09 34				
								92 50 30	267 09 32				
								92 50 32	267 09 28				
		1	170.015	-12.125	169.585	94 05 26				50 03 18		1.65	
		2	200.689	-10.220	200.689	92 55 00				46 33 29		1.65	
		3	212.457	-9.753	212.233	92 37 58				42 12 18		1.65	
		4	209.185	-10.092	208.941	92 46 02				38 03 50		1.65	
		5	70.384	-25.077	65.765	110 52 22				30 15 54		1.65	
		6	94.214	-33.286	88.137	110 41 30				270 55 04		1.65	
		7	122.639	-34.872	117.575	106 31 22				261 58 00		1.65	
		8	21.598	-2.193	21.486	95 50 00				165 23 36		1.65	
		9	38.993	-2.205	38.921	93 14 30				111 13 10		1.65	
		10	193.369	-38.319	189.534	101 25 46				253 20 00		1.65	
		11	136.610	-19.493	135.212	98 12 14				212 43 00		1.65	
		12											
		13											
		14											

Table B-5: FIELD SURVY OBSERVATION TABLE

						Zenith Angle (B.S) ° ' "		Zenith Angle (F.S) ° ' "		Horizontal Angle ° ' "			
Back sight	**Occupied**	**Fore sight**	**Slope Distance**	**Elevation Difference**	**Horizontal Distance**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Instrument Height**	**Reflector Height**
	ST 105												
ST 104			1076.198	-9.179	1076.160	90 29 20	269 30 38			150 49 14	150 49 12	1.65	1.65
						90 29 22	269 30 40			150 49 12	150 49 10	1.65	1.65
						90 29 18	269 29 42			150 49 16	150 49 14	1.65	1.65
		ST 106	1606.158	-47.54	1605.449			91 41 46	268 18 12				
								91 41 44	268 18 10				
								91 41 42	268 18 14				
		1	120.534	-11.581	119.968					19 49 22		1.65	
		2	205.255	-20.235	204.284					75 45 56		1.65	
		3	589.552	-43.88	587.976					118 09 38		1.65	
		4	606.197	-42.617	604.697					117 37 08		1.65	
		5	662.425	-54.092	660.256					129 42 05		1.65	

LIST OF TABLES

Methodology of assessment

We depend on standard and specification to put the important points, which is must taken to deal with a study area.

Select closed traverse to determine the location of the checkpoints.

After the field survey work is finished, we put the outlines to how, to deal with assessment procedure.

- ✓ Primary design of $\pm\sim$,$\pm_d$ (using error propagation techniques in angle and in distance).

1. Rreading error $\pm\sim_r = \frac{\pm r\sqrt{2}}{\sqrt{n}}$.
2. Eerrors in pointing $\pm\sim_p = \frac{\pm p\sqrt{2}}{\sqrt{n}}$.
3. Eerrors in target $\pm\sim_t = \frac{\sqrt{D_1^2 + D_2^2}}{D_1 D_2}\pm t^{\circ}$.
4. Eerrors in the instrument centering $\pm\sim_i = \frac{D_3}{D_1 D_2\sqrt{2}}\pm i^{\circ}$.

Errors in electronic distance measurements

$$\pm_D = \sqrt{\pm_i^2 + \pm_t^2 + a^2 + (D * b_{ppm})^2}$$

- ✓ Classify the order of checkpoints (locally and network according to FGCC)
- ✓ Determine the angular and linear closure (using error propagation in traverse survey).

Order class	First	Second		Third	
		I	II	I	II
Azimuth closure at Azimuth checkpoint	$1.7\sqrt{N}$	$3\sqrt{N}$	$14.5\sqrt{N}$	$10\sqrt{N}$	$12\sqrt{N}$
Position closure after Azimuth adjustment	$0.04\sqrt{K}$ OR *1 : 100000*	$0.08\sqrt{K}$ OR *1 : 50000*	$0.2\sqrt{K}$ OR *1 : 20000*	$0.4\sqrt{K}$ OR *1 : 10000*	$0.8\sqrt{K}$ OR *1 : 5000*

- ✓ Adjustment horizontal surveys traverse (least square solution).
- ✓ Error ellipses to describe the quality to the points.
- ✓ Compare the coordinates of existing map with the computing coordinates from field survey.
- ✓ Use (FSCE, FGDC, NMAP) standards to designate the accuracy of the map.

Survey contain

1. control points
2. unknown stations
3. distance observations
4. angle observation

- Control points: coordinate are fixed, and $\pm_x, \pm_y$ are zero.
- Unknown stations: the are computed depending on the control points.
- Distance observation and their standard deviation are obtained initial by error propagation.
- Angle observation and their estimated error (standard deviation) are obtained initial by error propagation.

Depending on these information we can make the following:

1. determine the linear and angular misclosure.
2. adjust the survey using least square solution (matrix program) to get:
   - residual for angle and distance observations.
   - Standard deviation for angle and distance observations.
   - Reference standard deviation $S_\circ$.
   - Covariance matrix Q_{xx}.
3. depending on part 2 we adjust the coordinate and error ellipses.
   - Estimated error for adjusted coordinate using

$$S_x = S_\circ\sqrt{q_{xx}}$$

$$S_y = S_\circ\sqrt{q_{yy}}$$

   - Error ellipses computed for point as:

$$S_u = S_\circ\sqrt{q_{uu}}$$

$$S_v = S_\circ\sqrt{q_{vv}}$$

$$tan(2t) = \frac{2q_{xy}}{q_{yy}\ q_{xx}}$$

$$\pm^2 u = \pm^2 x \sin^2(t) + 2\pm xy\cos(t)\sin(t) + \pm^2 y\cos^2(t)$$

$$\pm^2 v = \pm^2 x \cos^2(t) - 2\pm xy\cos(t)\sin(t) + \pm^2 y\sin^2(t)$$

The semimajor axis = $\pm u$ = a = $\sqrt{\pm^2 u}$

The semiminor axis = $\pm v$ = b = $\sqrt{\pm^2 v}$

Methodology of the calculation

1. Compute $\pm x, \pm y$ for all traverses points by least square solution.
2. Compute $\pm x, \pm y$ for any checkpoint referred to traverses points by writing the observation equation of latitude and departure:

$$X = d * \sin A_Z \qquad (1)$$

$$Y = d * \cos A_Z \qquad (2)$$

3. Check for the order of survey, relative error ellipses:

Construct all possible pairs for the checkpoints and the available Geodetic Network Points (GNP).

In our case the $\pm x_{points}, \pm y_{points}$ for the GNP are not available. Then may

- Assume $\pm x_{points}, \pm y_{points}$ for GNP as zero.
- Construct the pairs with the existing traverse points.

4. Calculate $\pm_{dx}, \pm_{dy}$ by using the pairs and the following equation:

$$d = \sqrt{(x_1 - x_2)^2 + (y_1 - y_2)^2}$$

5. Compute $\pm_{dv}, \pm_{du}$ based on $\pm_{dx}, \pm_{dy}$.
6. Compute the circular error according to the case:

The ratio $\pm v / \pm u$ =a/b determines the relationship used to calculate $\pm c$.

- When $\pm v / \pm u$ is between 1.0 and 0.6, then.

$$\pm c = 0.5222 \pm v + 0.4778 \pm u \qquad (a)$$

- A good approximation that yields as lightly larger $\pm c$ (i.e. on the safe side) is given by:

$$\pm c = 0.5(\pm v + \pm u) = 0.5(a + b) \qquad (b)$$

Which may be extended to the limit of $\pm$ min/$\pm$ max $= \pm v / \pm u \leq 0.2$

- Instead of equation (c) we can use:

$$\pm \bar{c} = \left[0.5(\pm^2 c + \pm^2 y)\right]^{1/2} \qquad (c)$$

This equation is applied only when $\pm v / \pm u$ is between 1.0 and 0.8, in which case it yields essentially the same value of $\pm c$ as in equation c.

7. Compute $\pm_c$ at certain level of confidence 95%

$$\pm c_{95\%} = 2.447 * \pm c$$

8. Compute the relative precision.

$$a = d / \pm c_{95\%}$$

compare this value with **table (1)**

classification	Minimum distance accuracy (a)
First order	1:100000
Second order, class I	1:50000
Second order, class II	1:20000
Third order, class I	1:10000
Third order, class II	1:5000

9. consider the worst pairs.
10. classify the work to the worst pairs.

3.2 Triangulation

Triangulation is a measurement system comprised of joined or overlapping triangles of angular observations supported by occasional distance and astronomic observations. Triangulation is used to extend horizontal control

Network Geometry

Order	First	Second	Second	Third	Third
Class		I	II	I	II
	15	10	5	0.5	0.5

Station spacing not less

than(kn)

Avwuge nns~nuun d~ame

eaglet of r~m r~

lmsthan — 40 35 30 300 25 Mtamum distanoranglet

of all r~ one

lusthan ...... 30 25 2S 20 20 Base line spacing nor

~ethmn(triang1a) S 10 12 15 15 Asuoonmic azimuth spacing not more

than(triangles) — 8 10 10 12 15

t DEasszattu~ b a*m 6~ thongh wtEb ~ana ~

The new survey is required to tie to at least four network control points spaced well apart. These network points must have datum values equivalent to or better than the intended order (and class) of the new survey. For example, in an arc of triangulation, at least two network control points should be occupied at each end of the arc. Whenever the distance between two new unconnected mzzvey points 5 1 than ~) percent of the distance between those points traced along existing or new connections, then a direct connection should be made between those two survey points. In addition, the survey should tie into any sufficiently accurate network control points within the station ipacing distance of the survey. These network stations should be occupied and sufficient observations taken to make these stations integral parts of the survey. Nonredundant geodetic connections to the network stations are not considered sufficient ties. Nonredundantly

determined stations are not allowed. Control stations should

not be determined by intersection or resection methods.

Simultaneous reciprocal vertical angles or geodetic leveling are observed along base lines. A base line need not be

observed if other base lines of sufficient accuracy were

observed within the base line spacing specification in the network, and similarly for astronomic azimuths.

Instrumentation

Only properly maintained theodolites are adequate for observing directions and azimuths for triangulation. Only precisely marked targets, mounted stably on tripods or supported towers, should be employed. The target should have a clearly defined center, resolvable at the minimum control spacing. Optical plummets or collimators are required to ensure that the theodolites and targets are centered over the marks. Microwave-type electronic distance measurement (EDM) equipment is not sufficiently accurate for measuring higher-order base lines.

Order	Firs:	Second	Second	Third	Third
Class		I	II	I	II
Thcodohte, least count	0.2’	0.2”	1.0	1.0”	1.0”

Calibration Procedures

Each year and whenever the difference between direct and reverse readings of the theodolite depart from 1 80 by more than 30, the instrument should be adjusted for collimation error. Readjustment of the cross hairs and the level bubble should be done whenever their mis-adjustments affect the instnment reading by the amount of the least count.

All EDM devices and retroreflectors should be serviced regularly and checked frequently over lines of known distances. The National Geodetic Survey has established specific calibration base lines for this purpose. EDM instruments should be calibrated annually, and frequency checks made semiannually.

Field Procedures

Theodolite observations for first-order and second-order, class I surveys may only be made at night. Reciprocal vertical angles should be observed at times of best atmospheric conditions (between noon and late afternoon) for all orders of accuracy. Electronic distance measurements need a record at both ends of the line of wet and dry bulb temperatures to ± I ‘C. and barometric pressure to ± 5 mm of mercury. The theodolite and targets should be centered to within 1 mm over the survey mark or eccentric point.

Order Firs: Second Second Third Third

Class I II I II

Nurnbcr of positions 16 16 8 or 12t 4 2

Order First Second Second Third Third

Class I II I II

Standard deviation of

mean not to exceed 0.4” 0.5” 0.8’ 1.2’ 2.0”

Rejection limit from

themean 4~• 4•• 5•• 5•• 5••

25

2

Reciprocal Vertical Angles

(along distance sight path)

Number of independent observations

direct/reverie 3 3

Maximum spread 10” 10”

Maximum time interval

between reciprocal

angles(hr)

Astronomic Azlmutlu

Observations per night 16 16

Number of nights 2 2

Standard deviation of

mean not to exceed 0.45” 0.45”

Rejection limit from

the mean S

Thctro-Opdcal Distances

Minimum number of days..

Minimum number of

measurements/day25

Minimum number of con- centric observations/ measurement 2

Minimum number cf offset observations/ measurement 2

Maximum difference from mean of observations (mm) 40

Minimum number of readings/observation (or equivalent) 10

Maximum difference from mean of readings (mm)..

Infrared Distances

Minimum numberofdays.. —Minimum number of measurements

Minimum number of concentric observations!

measurement

Minimum number of offset

obsereataons/

measurement

Maximum difference from

mean of observations

(mm)

Minimum number of

readings/observation

(or equivalent)

Maximum difference from

mean of readings (mm).. —Microwave Distances

Minimum number of

measurements

Minimum time span

between measurements

(hr)

2 2 2

I0’•10’28”

I I I

16 8 4

1 1 1

0.6” 1.0” 1.7

5” 6” 6”

25

I I

2 2 1 1

40

5060 60

10 10 10 10

* t *

2 1 1 1

25 25 1 1

1 1 1 1

2 1 1 1

5 5 10 10

10 10 10 10

* : : *

— 2 1

— — S —

Order First Second Second Third Third

Class I II I II

Maximum difference

between measurements

(mm) — — —100—

Minimum number of con-
centric observations!
measurement — —— 2~

Maximum difference from
mean of observations
(mm) — — —100ISO

Minimum number of
readings/observation
(or equivalent) — —— 20 . 20

Maximum difference from

meanofreadings(mm) — — —

S iffl.2”. 12 if .0” resolution.

two or more imtruments.

5 one measurement at each end of the line.

as specified by manufacturer.

~eamod out at both ends of the line.

Measurements of astronomic latitude and longitude are not required in the United States, except perhaps tar first-order work, because sufficient information for determining deflections of the vertical exists. Detailed procedures can be found in Hoskinson and Duerksen (1952).

Office Procedures

Order First Second Second Third Third

Class I Ii I II

Triangle Clcstre

Average not to exceed 1.0’ 1.2” 2.0 3.0” 5.0”

Maximum not to exceed 3” 3” 5 5 10”

Side Checks

Mean absolute correction

by side equation not

to exceed0.3’0.4 0.6 0.52.0”

A minimally constrained least squares adjustment will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation in this correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models which account for the following:

semimajor axis of the ellipsoid (a — 6378137 as)
reciprocal flattening of the ellipsoid (1/f — 298.257222)
mark elevation above mean sea level (known to ± I as)
gwid heights (known to ±6 as)
deflecxions of the vertical (known to ± 3)
geodesic correction
skew normal correction
height of instrument

height of target
sea level correction

arc correction

teoid height correction

s~nd velocity correction crustal motion

3.3 Traverse

Traverse is a measurement system comprised of joined distance and theodolite observations supported by occasional

astronomic observations. Traverse is used to density horizontal control.

Network Geometry

Order First Second Second Third Third

Class I Ii I II

Station spacing not less

than(ktn) 10 4 2 0.5 0.5

Maximum deviation of

main traverse from

straight line 20 20 25 30 400 Minimum number of

benchm.a.rkties 2 2 2 2 2 Bench mark tie spacing

not more than

(seynenta) 6 S 10 15 20 Astronomic azimuth

spacing not moms than

(segments) 6 12 20 25 40 Minimum number of

networkcoetxrolpoints 4 3 2 2 2

The new survey is required to tie to a minimum number of network control points spaced well apart. These network points must have datum values equivalent to or better than the intended order (and class) of the new survey. Whenever the distance between two new unconnected survey points is less than 20 percent of the distance between those points traced along existing or new connections, then a direct connection must be made between those two survey points. In addition, the survey should tie into any sufficiently accurate net~k control points within the station spacing distance of the survey. These tics must include EDM or taped distances.

Nottredundant geodetic connections to the network stations are not considered sufficient ties. Nonredundantly determined stations are not allowed. Reciprocal vertical angles or geodetic leveling are observed along all traverse lines.

lmtnimentation

Only properly maintained theodolites are adequate for observing directions and azimuths for traverse. Only precisely marked targets, mounted stably on tripods or supported towers, should be employed. The target should have a clearly defined center, resolvable at the minimum control spacing. Optical plummets or collimators are required to ensure that the theodolites and targets are centered over the marks. Microwave-type electronic distance measurement equipment is not sufficiently accurate for measuring fist-order traverses.

Order	First	Second	Second	Third	Third
Class		I	II	I	II
Thcodolite, least count	0.2	1.0	1.0	1.0	1.0

Calibration Procedures

Each year and whenever the difference between direct and reverse readings of the theodolite depart from 180 by more than 30,. the instrument should be adjusted for collimation error. Readjustment of the cross hairs and the level bubble should be done whenever their misadjustments affect the instrument reading by the amount of the least count.

All electronic distance measuring devices and retroreflectors should be serviced regularly and checked frequently over lines of known distances. The National Geodetic Survey has established specific calibration base lines for this purpose. EDM instruments should be calibrated annually, and frequency checks made semiannually.

Field Procedures

Theodolite observations for first-order and second-order, class I surveys may be made only at night. Electronic distance measurements need a record at both ends of the line of wet and dry bulb temperatures to ± 1 ‘C and barometric pressure to ± 5 mm of mercury. The theodolite, EDM, and targets should be centered to within 1 mm over the survey mark or eccentric point.

Order First Second Second Third Third

Clans I II I II

Dfrse~

Numberofpamitions 16 Sos 12t 6orV 4 2

Standard deviation of mean

not to exceed 0.4”, 0.5’ 0.5” 1.2’ 2.0”

Rejectiouslimitfrtttntjsemean 4•. 5•~ 5 5•• 5••

Reciprocal Vutlcal Aagiua

(aiming dhtamm dght ~th)

Number of independent

obaervationsdirect/revenue 3 3 2 2 2

Maximumspread10”10’10”10” 20”

Maximum tine itnanl between

reciprocalangles(hr) 1 1 1 1 1

~ Am̊~bs

Obaervatiomtspernight16 1612 S 4

Numberof sights 2 2 1 1 1

Standard deviation of meats

not to exceed 0.45” 0.45~ 0.6” 1.0” 1.7”

Rejectionlimitfromthemean.... 5~• 5~ 5~• 6” 6”

~

Minimum number of

measurements1 1 1 1 1

Minimum number of concenttic obaervatiomu/mea.surement1 1 1 1 1

Minimum number of offset obsetvatiom/meaurement

Maximum difference from mean of observations (mm)60 60 — — —

Order Firs: Second Second Third Third

a~s i it t II

Minimum number of rtadings/

obscnation(oreqwvalest)1010 10 10 10

Maximum difference front

mcanofreadinp(mm) I I I I I

~

Minimum number of

measurements1 1 1 1

Minimum number of concentric obeervatioau/minswement 1 1 1 1 1

Minimum number of offset obser'.'atiora/measurtment 1 1 1 — —

Maximum diltetetscc frosts

mcanofobmenaticns(mm) 10 10 1(4 — —

Minimum number of readings/

observation 10 10 10 10 10

Maximum dilTcrence from

momsofrmdinp(mm) I 1 1 £ 3

Mlaowav. Dhsn

Minimum number of

measurements — 1 1 1 1

Minimum number of concentric

observatiosu/measunment — 2~ 1, l~ 1~

Maximum difference from

mcanofobservatioru(mm)—150 150 200 200

Minimum number of rudings/

observation —20 20 10 10

Maximum diff from

meanofrmdinp(mm)— I I I £

tSif 02”, l2lfl.0”twah.tiom.

61f0.2”.Sifl.O”ruaIutiut.

5as specified by sufscturer.

ly if deeinul rWdit~ sun Oar high 9’.

mat at beth the Ii...

Measurements of astronomic latitude and longitude are not required in the United States, except perhaps for first-order work, because sufficient information for determining deflections of the vertical exists. Detailed procedures osn be found in Hciekinson and Duerksen (1952).

Office Pmceàru

Order First Second Second Third Third

Cla.r.r I -II I II

Azimuth closure at azimuth

check

(satxmàdarc). 1.7s/N 3.0~/N 4.5VN 10.O~/N 12.OV'N Position closure 0.O4VK 0.OSv/K 0.201K o.4OVK o.5OVK

after azimuth ... or or or or or

adjustmentf 1:100,000 liO,000 120,000 1:10,000 li,000

(N it tmmnbaraf a~u. Eu rusts dbtanat I. Las)

lbs p.u~s meaitEtg the square rust it duigud far ~ whose higher praparitwul ameuey a inquired. Lion the fmutuls then pm'. lbs unsllut permu.iblc tissue. The closure (e.g., 1:100,000) a obtained by wutputing the ar..~ isus thqu~uai l~ wha~ ud tBmiم~ tbsiffuutm by K.

Nate.' Do ~ ftin tissue with detests acaracy tithe nuwey.

A minimally constrained least squares adjustment will be checked for blunders by examining the normalized residuals. The observation weights will be checked by

inspecting the postadjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation in a correctly weighted least squares adjustment will indicate the provisional accuracy classification, A survey variance factor ratio will be computed to check

for systematic error. The least squares adjustment will use models which account for the following:

semimajor axis of the elliyaoid (a — 6378137 m)

reciprocal flattening of the elliwoid (I/f — 298.257222)

mark elevation above mean sea level (known to ± I m)

geoid heights (known to ± 6 m)

deflectioru of the vertical (known to ±31 geodesic correction

akew normal correction height of izutrument

height of target

sea level correction arc correction

geoid height correction

second velocity correction

crustal motion

3.4 Inertial Surveying

Inertial surveying is a measurement system comprised of lines, or a grid, of Inertial Surveying System (ISS) observations. These specifications cover use of inertial systems only for honrnntal control,

Network Geometry

Order	Second	Second	Third	Third
Class	I	II	1	II
Stationspecingnotlensthan(jcm)..,	10	4	2	I
Maximum deviation from straight line connecting endpoints	20	250	300	350

Each inertial survey line is required to tie into a minimum of four horizontal network control points spaced well apart and should begin and end at network control points. These network control points must have horizontal datum values better than the intended order (and class) of the new survey. Whenever the shortest distance between two new unconnected survey points is less than 20 percent of the distance between those points traced along existing or new connections, then a direct connection should be made between those two survey points. In addition, the survey should connect to any sufficiently accurate network control points within the distance specified by the station spacing. The connections may be measured by EDM or tape traverse, or by another ISS line. If an ISS line is used, then these lines should follow the same specifications as all other 155 lines in the survey.

For extended area surveys by 155, a grid of intersecting lines that satisfies the 20 percent rule stated ahove can be designed. There must be a mark at each intersection of the lines. This mark need not be a permanent monument; it may be a stake driven into the ground. For a position to

receive an accuricy classification, it must be permanently monuxnented.

A grid of intersecting lines should contain a minimum

of eight network points, and should have a network control point at each corner. The remaining network control

points may be distributed about the interior or the periphery

of the grid. However, there should be at least one network

control point at an intersection of the grid lines near the

center of the grid. If the required network points are not available, then they should be established by some other measurement system. Again, the horizontal datum values of these network control points must have an order (and class) better than the intended order (and class) of the new survey.

Immmuentauoa

155 equipment falls into two types: analytic (or strapdown) and semianalytic. Analytic inertial units are not considered to possess geodetic accuracy. Semianalytic units are either “space stable” or “local level.” Space stable systems maintain the orientation of the platform with respect to inertial space Local level systems continuously torque the accelerometers to account for Earth rotation and movement of the inertial unit, and also torque the platform to coincide with the local level. This may be done on command at a coordinate update, or whenever the unit achieves ~ro velocity (Zero velocity UPdaTe, or “WIT’). Independently of the measurement technique, the recorded data may be filtered by an onboard computer. Because of the variable quality of individual ISS instruments, the user should test an instrument with existing geodetic control beforehand.

An offset measurement device accurate to within 5 mm should be affixed to the inertial unit or the vehicle.

Calibration Procedures

A static calibration should be performed yearly and immediately after repairs affecting the platform, gyroscopes, or accelerometers.

A dynamic or field calibration should be performed prior to each project or subsequent to a static calibration. The dynamic calibration should be performed only between horizontal control points of first-order accuracy and in each cardinal direction. The accelerometer scale factors from this calibration should be recorded and, if possible, stored in the onboard computer of the inertial unit.

Before each project or after repairs affecting the offset measurement device or the inertial unit, the relation between the center of the inertial unit and the zero point of the offset measurement device should be established.

Field Procedures

When surveying in a helicopter, the helicopter must come to rest on the ground for all ZUPT’s and all measurements.

OrderSecond Second Third Third

Class I II I II

Minimum number of complete

runsperline 2 1 1 1

Maximum deviation from a uniform rate of tsavet

(including ZUPT) 15%20%25% 30%

Maximum ZIJPT interval (ZUPT toZtjFr)(sec) 200240300300

A complete ISS measurement consists of measurement of the line while traveling in one direction, followed by measurement of the same line while traveling in the re verse direction (double-run). A coordinate update should not be performed at the far point or at midpoints of a line, even though those coordinates may be known.

The mark offset should be measured to the nearest 5 mm.

Office Procedures

Order	Second	Second	Third	Third
Class	I	II	I	II
Maximum difference of umoothcL~ coordinates between forward andrever,crun(cm)	60	60	70	80

A minimally constrained least squares adjustment of

the raw or filtered survey data will be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight Distance standard errors computed by error propagation in this correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use the best available model for the particular inertial system. Weighted averages of individually smoothed lines are not considered substitutes for a canbined least squares adjustment to achieve geodetic accuracy.

3.5 Geodetic Leveling

Geodetic leveling is a measurement system comprised of elevation differences observed between nearby rods. Leveling is used to extend vertical control.

Network Geometry

Order	First	First	Second	Second	Third
Class	I	II	I	II	
Bench mark spacing not morethan(km)	3	3	3	3	3
Average bench mark spacing notmorethan(k.rn)	1.6	1.6	1.6	3.0	3.0

Order	First	First	Second	Second	Third
Class	I	II	I	II	
Line length between network control points not more tltn(kzn)	300	100	50	50 (double-nan) 25 (single-run)	2.5 10

New surveys are required to tie to existing network bench marks at the beginning and end of the leveling line. These network bench marks must have an order (and class) equivalent to or better than the intended order (and class) of the new survey. F.rst-order surveys are required to perform check connections to a minimum of six bench marks, three at each end. All other surveys require a minimum of four check connections, two at each end. "Check connection" means that the observed elevation difference agrees with the adjusted elevation difference within the tolerance limit of the new survey. Checking the elevation difference between two bench marks located on the same structure, or so close together that both may have been affected by the same localized disturbance, is not considered a proper check~ In addition, the survey is required to connect to any network control points within 3 km of its path. However, if the survey is run parallel to existing control, then the following table specifies thc maximum spacing of extra axmoctkms between the survey and the control. At least one extra connection should always be made.

Distance. stuwy to network	Maximum spacing of extra cotuwctians (kin)
O.Sknsoricu	.. - S
O.Skmto2.Okzn	sO
2.Okmto3.Okm	20

LuUimnmtadoa

Order	First	First	Second	Second	Third
Class	I	II	I	II	
Minimum repeatability of line of sight	025'~	0.25'	050~	0.50"	1.00"

Leveling rod construction	IDS	IDS	IDSt	ISS	Wood	or
uwlSS	Metal					

Inmonuzumt mmd sad ruohdoin

Lcastcount(mm)	0.1	0.1	0.5-1.0’	1.0	1.0

(IDS-tnvar, dmable semi.)

(~-tnvar. si.sh semi.)

if~tiaml mianm.sar u used.

‘1.0mm! 3-.iue method, 0.5mm! oçsieal miaemneoer.

Only a compensator or tilting leveling instrument with an optical micrometer should be used for first-order leveling. Leveling rods should be one piece. Wooden or metal rods may be employed only for third-order work. A turning point consisting of a steel turning pin with a driving cap should be utilized. If a steel pin cannot be driven, then a turning plate (“turtle”) weighing at least 7 kg should be substituted. In situations allowing neither turning pins nor turning plates.(sandy or marshy soils), a long wooden stake with a double-headed nail should be driven to a firm depth.

Calibration Procedures

Order	First	First	Second	Second	Third

Class I II I II

Lew~~

Maximum collimation error,

single line of sight (mm/in) 0.05 0.05 0.05 0.05 0.10

Maximum collimation error,
reversible compensator type
instruments, mean of two
lines of night (mm/rn) 0.02 0.02 0.02 0.02 0.04

Taint interval between eeiliniation

error determinations not

longer than (days)

Reversiblecompensator 7 7 7 7 7 Othertypes 1 1 1 1 7

Maximum angular difference

between two lines of sight,

reversible compensator40” 40’ 40” 40” 60’

Iiwlbsgrod

Minimum scale calibration

ntandard N N N M M

Time interval between scale calibrations (yr)1 1 -

Leveling rod bubble verticality maintained to within10’10’10’ 10’ 10’

(N—National standard)

(M—Manufacturer’s standard)

Compensator-type instruments should be checked for proper operation at least every 2 weeks of use. Rod calibration should be repeated whenever the rod is dropped or damaged in any way. Rod levels should be checked for proper alignment once a week. The manufacturer’s calibration standard should, as a minimum, describe scale behavior with respect to temperature.

Field Procedures

Order First First Second Second Third

Class I II I II

Minimal observation

method micro- micro. micro 3-wire center metermeter meter or wire

3-wire

Section running SRDS SRDS SRDS SRDS SRDS

orDRorDR orDRt orDR orDR~

orSPorSP orSP

Field Procedtmts—Continued

First First Second Second Third

I II I II

5 10 10

10 10 10

60 70 90

0.5 0.50.5

Difference of forward and bsckward sight lengths

never to exceed

persetup(m) 2 5

persection(m) 4 10

Maairnum night length (in) 50 60

Mbsmum — ctannee

oflineofsight(m) 0.5 0.5

Even number of setups

when not using leveling

reds with detailed

calibration yes yes

Determine temperature gradient for the vertical range of the line of sight ateachnetupyes yes yes — —

Ma.timum section miscloture (mm) 3 ~/D 4VD 6i/D S ~/D l2~~/D

Minimum loop mssclwure(mm) 4VE 5~/E 6VE 8VE I2VE

yes yes—

— .~e Reverse direction of single

ruin every half day yea

kveilssg imstninsmnts

Off-level/relevel

umrument between observing the high

andlowrodscales yes

3~ .ethed

Reading chock (difference between top and ~toen intervals) for one setup net to exceed (tenths of rod units) ..

Read rod 1 first in

alternate setup method ... —1~e smle visé

l..ow-high scale elevation difference for one setup ~ to exceed (mm) With reversible

compensator 0.40

o~ s~~i ~ee

Half-oentsmetee rods 0.25

Full-centimeter rods ... 0.30

yes yes - -

yes yen - -

— 2 2 3

— yea yes yes

1.00 1.00 2.00 2.00

0.30 0.60 0.70 1.30

0.30 0.60 0.70 1.30

(SRDS—Single-Run, Disable Sinsultanuaun pnewdure)

(DR—Disable-Run)

(SP—SPsar, 1 thin 25 km. dnsbl.mu)

D’—hortent length duetian (wswwsy) in kin

E—peruneter of leap us kin

Must d.sblequn when using Swim method.

• May single-run if line length between network control points is loss than 25 bit

I May single-ran if line length between network control points is loss than 15 ks,s.

Double-run leveling may always be used, but single-

run leveling done with the double simultaneous procedure may be used only where it can be evaluated by loop closures. Rods should be leap-frogged between setups

(alternate setup method). The date, beginning and ending

times, cloud coverage, air temperature (to the nearest

degree), temperature scale, and average wind speed should be recorded for each section plus any changes in the date, in~trumentation, observer or time zone. The instrument need not be off-leveled/releveled between observing the high and low scales when using an instrument with a reversible compensator. The low-high scale difference tolerance for a reversible compensator is used only for the control of blunders.

With double scale rods, the following observing sequence should be used:

becksight, low-scale bscksight, stadia

foresight, low-scale foresight. stadia

off-level/relevel or reverse compensator

foresight. high.ocale

backsight. high-scale

Office Proculwu

Order First First Second Second Third

Claus I II I II

~d ~dmu

(backward and forward)

Algebraic sum of all

corrected section misclwwu

of a leveling line

nottoexceed(mm) 3VD 4VD 6~/D S~/D 12~/1)

Section misclo.ure not to exceed(mm) 3v/E 4v’E 6VE SVE I2VE

Loap

Algebraic sum of all

corrected ntisckaures

nottoexcoed(mm) 4t/F S~’F 6VF 8~/F 12VF Loop msnclouure not

toexcoed(mm) 4~/F 5VF 6VF SVF l2~/F

(D—uhuiat length of leveling km (ass-way) in kin)

(E—.honat ass-way length of suetion iii kin)

(F—length of hasp in kin)

The normalized residuals from a minimally constrained

least squares adjustment will be checked for blunders. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Elevation difference standard errors computed by error prvpegatian in a axrectly waighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models that armunt fot

grav,ty effect or orthometric correction rod scale errors

rod (lnvar) temperature

refraction—n~ latitude and longitude to 6 or venial temperature difference observations between 0.5 and 2.5 en above the earth tides and magnetic field

collimation error

crustal motion

3.6 Photogrammetry

Photogrammetry is a measurement system comprised of photographs taken by a precise metric camera and measured by a comparator. Photogrammetry is used for densiflcation of horizontal controL The following specilications apply only to analytic methods.

Network Geometry

Order	Second	Second	Third	Third
Claus	I	II	I	II
Forward overlap not Ins than	66%	66%	60%	60%
Side overlap not Ins than	66%	66%	20%	20%
Intersecting rayn per ~3int not Iasthan(designcriteria)	9	5	3	3

The photogrammetric survey should be areal: single strips of photography are not acecptable~ 11~ survey should encompass, ideally, a minimum of eight horizontal control points and four vertical points spaced about the perimeter of the survey. In addition, the horizontal control points should be spaced no farther apart than seven air bases. The horizontal control points should have an ~er (and class) better than the intended order (and dam)

of the survey. The vertical points need not meet geodetic control standards. If the required control points are not available, then they must be established by some other measurement system.

Lubwnmtedoa

Order	Second	Second	Third	Third
Class	I	II	I	II

Meeic Cmi

Maximum warp of platen not

morethan(,am)- 10 10 10 10

Dimensional oousuol not

kmthan meats 5 3 3

with fiduclals fiducial. fiduciala

maximum

of 2 an

Iastcount(jamn) 1 1 1 1

The camera should be of at least the quality of those employed for large-scale mapping. A platen should be included onto which the film must be satisfactorily flattened during exposure. Note that a reseau should be used for second-order, class I surveys.

Calibradom Procedwa

Order	Second	Second	Third	Third
Class	I	II	I	II
Metric aeu				
Root mean square of calibrated radial distortion not more than~m)	1	3	3	s

Calibration Procedures—Continued

Order	Second	Second	Third	Third
Claus	I	II	I	II

Root mean square of calibrated

decentering distortion not more

than(ism) I 5t 5t 51

Root mean square of reacau

ooordinatesnotmorethanUam) 1 1 3 3

Root mean square of fiducial

ooordinatcsnotmorethan(jzrn) — 1 3 3

not usually treated separntcly itt earners ealibratton facilities manufacturer’s cerufleatioe is satisfactory.

The metric camera should be calibrated every 2 years, and the comparator should be calibrated every 6 months. These instruments should also be calibrated after repair or modifications.

Characteristics of the camera’s internal geometry (radial symmetric distortion, decentered lens distortion, principal point and point of symmetry coordinates, hnd reseau coordinates) should be determined using recognized calibration techniques, like those described in the current edition of the Manual of Photogrammetry. These characteristics will be applied as corrections to the measured image coordinates.

Field Procedures

Photogrammetzy involves hybrid measurements a meuic camera photographs targets and features in the field, and a comparator measures these photographs in an office environment. Although this section is entitled “Field Procedures,” it deals with the actual measurement process and thus includes comparator specifications

Order	Second	Second	Third	Third
Claus	I	II	I	II
Targets				
Control points targeted	yea	yea	yea	yea
Pass points targeted	yea	yea	optional	optional
Coa.pareter				
Pointingspertargetnotlessthan	4	3	2	2
Pointings per reseau (or fiducial) notlessthan	4	3	2	2
Number of different reseau intersections per target not kssthan	4	—	—	—
Rejection limit from mean of pointingspertarget(sm)	3	3	3	3

Office Procedures

Order	Second	Second	Third	Third

Claus	I	II	I	II
Root mean square of adjusted photocoordinatea not more thanüsm)	4	6	3	12

A least squares adjustmrnt of the phot ocoordinates, constrained by the coordinates of the horizontal and vertical control points, will be checked for blunders by examining the normalized residuals- The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation in this correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models that incorporate the quantities determined by calibration.

11 Satellite Doppler Positioning

Satellite Doppler positioning is a three-dimensional measurement system based on the radio signals of the U.S. Navy Navigational Satellite System (NNSS), commonly referred to as the TRANSIT system. Satellite Doppler positioning is used primarily to establish horizontal control.

The Doppler observations are processed to determine station positions in Cartesian coordinates, which can be transformed to geodetic coordinates (geodetic latitude and longitude and height above reference ellipsoid). There are two methods by which station pouitiom can be derived. point positioning and relative positioning.

Point positioning, for geodetic applications, requires

that the processing of the Doppler data be performed with the precise ephemerides that are supplied by the Defense Mapping Agency. In this method, data from a single station is processed to yield the station coordinates.

Relative podtonng is pcndble when two or more receivers are operated together in the survey area. The processing of the Doppler data can be performed in four modes:

simultaneous point positioning, translocation, semishort arc, and shozt arc. The specificaticxu for relative positioning are valid only for data reduced by the semishort or short arc methods. The semishort arc mode allows up to 5 degrees of freedom in the ephemerides; the short arc mode allows 6 or more degrees of freedom. These modes allow the use of the broadcast ephemerides in place of the precise ephemerides.

The specifications quoted in the following sections are bused on the experience pined fmxn the analysis of Doppler surveys performed by agencies of the Federal government Since the data aze primarily from surveys perfonned within the continental United States, the precisions and related specifications may not be appropriate for other areas of the world.

Network Geometry

The order of a Doppler survey is determined by: the spacing between primary

Doppler stations, the order of the base network stations from which the primaries are established, and the method of data reduction that is used. The order and class of a survey cannot exceed the

lowest order (and class) of the base stations used to establish the survey.

The primary stations should be spaced at regular intervals which meet or exceed the spacing required for the desired accuracy of the survey. The primary stations will carry the same order as the survey.

Supplemental stations may be established in the same survey as the primary stations. The lowest order (and class) of a supplemental station is determined either by its spacing with, or by the order of, the nearest Doppler or other horizontal control station. The processing mode determines the allowable station spacing.

In carrying out a Doppler survey, one should occupy, using the same Doppler equipment and procedures, at least two existing horizontal network (base) stations of order (and class) squivalent to, or better than, the intended order (and class) of the Doppler survey. If the Doppler survey is to be first-order, at least three base stations must be occupied. If relative positioning is to be used, all base station base lines must be directly observed during the survey. Base stations should be selected near the perimeter of the survey, so as to encompass the entire survey.

Stations which have a precise elevation referenced br geodetic leveling to the National Geodetic Vertical Datum (NGVD) are preferred. This will allow geoidal heights to be determined. As many base stations as possible should be tied to the NGVD. If a selection is to be made, those stations should be chosen which span the largest portion of the survey.

If none of the selected base stations is tied to the NGVD, at least two~ preferably more, bench marl~ of the NGVD should be occupied. An attempt should be made to span the entire survey area.

Datum shifts for transformation of point position solutions should be derived from the observations made on the base stations.

The minimum spacing, D, of the Doppler stations may be computed by a formula determined by the prodessing mode to be employed. This spacing is also used in conjunction with established control, and other Doppler control, to determine the order and class of the supplemental stations.

By using the appropriate formula, tables can be constructed showing station spacing as a function of point or relative one-sigma position precision (s~ or s~) and desired survey (or station) order.

Point Positioning

D = 2V2s~a

where

a — denominator of distance accuracy classification standard (e.g., a — 100,000 for first-order standard).

Relative Positioning

First Second Second Third Third

I II I Ii

D (kin)

D —2 s_5a

D (kin)

200	566	242	114	56	28
100	283	141	57	28	14
70	200	100	40	20	10
50	141	71	26	14	7

a — denominator of distance accuracy classification standard (e.g., a - 100,000 for first-order standard).

50	100	50	20	10	5
35	70	35	14	7	4
20	40	20	8	4	2

where

However, the spacing for relative positioning should

not exceed 500 kin.

Imfrmnmmdon

The receivers should receive the two carrier frequencies transmitted by the NNSS. The receivers should record

the Doppler count of the satellite, the receiver clock times, and the signal strength. The integration interval

should be approximately 4.6 sec. Typically six or seven of these intervals are accumulated to form a 30-second Doppler axznt observation The reference frequency should be stable to within 5.0(10-”) per 100 sec. The maximum difference from the average receiver delay should not exceed 50 jasec. The best estimate of the mean electrical center of the antenna should be marked. This mark will be the reference point for all height-of-antenna measurements.

Calibradon Procedures

Receivers should be calibrated at least once a year, or whenever a modification to the equipment is made. It is desirable to perform a calibration before every project to verify that the equipment is operational. The two-receiver method explained next is preferred and should be used whenever possible.

Two-Receiver Method

The observations are made on a vector base line, of internal accuracy sufficient to serve as a comparison standard, 10 to 50 m in length. The base line should be located in an area free of radio interference in the 150 and 400 MHz frequencies. The procedures found in the table on relative positioning in “Field Procedures” under the 20 cm column heading will be used. The data are reduced by either shortarc or semishort arc methods. The receivers

First Second Second Third Third

I II I II

will be considered operational if the differences between the Doppler and the terrestrial base line components do not exceed 40 cm (along any coordinate axis).

Single-Receiver Method

Observations are made on a first-order station using the procedures found in the table on relative positioning in “Field Procedures” under the 50 cm column heading. The da4 are reduced with the precise ephemerides. The resultant position must agree within 1 m of the network position.

Field Procedures

The following tables of field procedures are valid only for measurements made with the Navy Navigational Satellite System (TRANSIT).

Point Positioning

s~ (precise ephemerides) .50cm 70cm 100cm 200cm

Max. standard deviation of mean of counts/pass (cm). broadcast ephemerides	23	25	25	23
Period of observation not less than (hr)	48	36	24	12
Number of observed passes not lesstltant	40	30	13	8
Number of acceptable passes (evaluated by on-site point processing) not less than	30	20	9	4
Minimum number of acceptable passes within each quadrant’	6	4	2	1
Frequency standard warm-up time (hr)				
CTyStaI	48	48	24	24
atomic	1.5	1.5	1.0	1.0
Maximum interval between metnorological observations (hr)	6	5	5	5

Number of passes refers to theme for which the precise ephemerides are available for reduction.

There should he a nearly equal number of northward and southward peas..

leach setup, visit and takodows.

Relative positioning

20cm 35cm 50cm

Maximum standard deviation of mean of

counts/pass (cm). broadcast ephemerides 25 25 25

Period of observation not less than (hr) 48 36 24

Numberofobservedpassesnotlessthant 40 30 13

Number of acceptable passes (evaluated by on-site point position processing) notlessthan 30 20 9

Minimum number of acceptable passes within each quadrant' 6 4 2

Frequency standard warm-up time (hr)

crystal 48 48 48

asomsc 1.5 1.5 1.5

Maximum interval between meteorological observations (hr) 6 6

Number of observed pan. refers to ill satellites available for truckis~ and reduction with the brsmdemst or precise ephemerides.

• Number of northward and southward peas, should be neaziy equal.

5 Each setup, visit and takedown.

The antenna should be located where radio interference is minimal for the 150 and 400 MHz frequencies. Meslium

frequency radar, high voltage power lines, transformers, excessive noise from automotive ignition systems, and high power radio and television transmission antennas

should be avoided. The horizon should not be obstructed above 7.5.

The antenna should not be located near metal structures, or, when on the roof of a building, less than 2 m from the edge. The antenna must be stably located within 1 mm over the station mark for the duration of the observations. The height difference between the mark and the reference point for the antenna phase center slxssid be measured to the nearest millimeter. If an antenna is moved while a pass is in progress, that pass is not acceptable. If moved, the antenna should be relocated within 5 mm of the original antenna height; otherwise the data may have to be processed as if two separate stations were established. In the case of a reoccupation of an existing Doppler station, the antenna should be relocated within 5 mm of the original observing height

Long-term reference frequency drift should be monitored to ensure it does not exceed the manufacturer’s specifications.

Observations of temperature and relative humidity should be collected, if possible, at or near the height of the phase center of the antenna. Observations of wet-bulb and dryboTh temperature readings should be rem,ded to the nearest 0.5C. Barometric readings at the station site should be recorded to the nearest millibar and corrected for difference in height between the antenna and barometer.

Office Pmcedwu

The processing constants and criteria for determining the quality of point and relative positioning results are as follows:

1. For all passes for a given Station occupation, the

average number of Doppler counts per pass should be at least 20 (before processing).

2. The cutoff angle for both data points and passes

should be 7.5.

3. For a given pass, the maximum allowable rejection

of counts, 3 sigma postprocessing, will be 10.

4. Counts rejected (excluding cutoff angle) for a solution should be less than 10 percent

5. Depending on number of passes and quality of data, the standard deviation of the range residuals for all passes of a solution should range between:

Point positioning—b to 20cm

Relative positioning—S to 20cm

A minimally constrained least squares adjustment will

be checked for blunders by examining the normalized residuals. The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight Distance standard errors computed by error propagation between points in this correctly weighted ieast squares adjustment will indicate the maximum achier-

able accuracy classification. The formula presented in

“Standards” will be used to amy at the actual classification.

The least squares adjustment will use rmxlels which acmunt

for~

tropcaphenc scale bias, 10 percent uncertainty receiver time delay

satellite/receiver frequency offset

precise ephemeris tropospheric refraction ionospheric refraction long-term ephemeris variations crustal motion

3.8 Absolute Gravimetry

Absolute gravimetry is a measurement system which determines the magnitude of gravity at a station at a

specific time. Absolute gravity measurements are used to

establish and extend gravity control. Within the context

of a geodetic gravity network~ as discu~i in “Standards,” a

series of absolute measurements at a control point is in

itself sufficient to establish an absolute gravity value for that location.

The value of gravity at a point is time dependent, being subject to dynamic effects in the Earth. The exte’flt of gravimetric stability can be determined only by repeated observations over many years.

Network Geometry

Network geometry cannot by systematized since absolute observations at a specific location are discrete and uncorrelated with other points. In absolute gravimetry, a network may consist of a single point

A first-order, class I station must possess gravimetric stability, which only repeated measurements can determine. This gravimetric stability should not be confused with the accuracy determined at a specific time. It is possible for a value to be determined very precisely at two different dates and for the values at each of these respective dates to differ. Although the ultimate stability of a point cannot be determined by a single observation seesion, an attempt should be made to select sites which are believed to be tectonically stable, and sufficiently distant from large bodies of water to minimize ocean tide a~stal loading.

The classification of first-order, class I is reserved for network points which have demonstrated long-term stability. To ensure this stability, the point should be rcobsetved at least twice during the year of establishment and thereafter at sufficient intervals to ensure the continuing stability of the point. The long-term drift should indicate that the value will not change by more than 20 gsGal for at least 5 years. A point intended as first’order, class I will initially be classified as first-order, class II until stability during the first year is demonstrated.

h.tnunmtsdon

The system currently being used is a ballistic-laser device and is the only one at the current state of technolo

gy considered sufficiently accurate for absolute gravity measurements. An absolute instrument measures gravity at a specific elevation above the surface, usually about 1 us. For this reason, the gravity value is referenced to that level. A measurement of the vertical gravity gradient, using a relative gravity meter and a tripod, must be made to transfer the gravity value to ground level. The accuracy of the relative gravimeter must satisfy the gravity gradient specifications found in "Field Procedures."

Calibration Procedwea

Ballistic-laser instruments are extremely delicate and each one represents a unique entity with its own characteristics. It is impossible to identify common systematic errors for all instruments. Therefore, the manufacturer's recommendations for individual instrument calibration should be followed rigorously.

To identify any possible bias associated with a particular instrument, comparisons with other absolute devices are strongly recommended whenever possible. Comparisons with previously established first-order, class I network points, as well as first-order, class II network points tied to the class I points, are also usefuL

F1e~ Procedwu

The following specifications were determined from results of a prototype device built by J. Faller and M. Zumberge (Zumberge~ M., “A Patable Apparatus for Absolute Measurements of the Earth’s Gravity,” Department of Physics, University of Colorado, 1981) and are given merely as a guideline. It is possible that some of these values may be inappropriate for other instruments or modeis Therefore~ exceptions to these specifications arc allowed on a case-by-case basis upon the recommendation of the manufacturer. Deviations from the specifications should be noted upon submission of data for ciassificatiois.

Order First First Second Third

Class I II

Ah~ .~

Standard dcviatitxs of each

— measurement set

nottoexoscd(uGal)20 20 50 100

Minimum number of sets/ observation ~

Maximum difference of a measurement set f ross mean of allmeasuremenus(oiGal)1212 37 43

Dai~etric prasun standard cmtr(mb.r) 4 4 — —

Gea1 .~

Standard deviation of measurement

of vertical gravity gradient at

time of observation (saGal/m) 5 5 5 5

Standard deviation of height of

iutrumcnt above point (mm) 1 1 5

Office Procedures

The manufacturer of an absolute gravity instrument

usually provides a reduction process which identifies and

accounts for error sources and identifiable parameters.

This procedure may be sufficient, making further office

adjustments unnecessary.

A least squares adjustment will be checked for blunders by examining the normalized residuala The observatitxs weights will be checked by inspecting the postadjustment estimate of the variance of unit weight. Gravity value standard deviations computed by error propagation in a

correctly weighted, least squares adjustment will indicate the provisional accuracy classification. The least squares adjustment, as well as digital filtering techniques and/or sampling, should use models which account for.

atmospheric mass attraction micrweismic activity instrumental characteristics

Iunisolar attraction

elastic and plastic response of the Earth (tidal leading)

3.9 Relative Gravimetry

Relative gravimetry is a measurement system which

determines the difference in magnitude of gravity between two stations. Relative gravity measurements are used to extend and densify gravity controL

Network Geometry

A first-order, class I station must possess gravimetric stability, which only repeated measurements can determine. This gravimetric stability should not be Confused with the accuracy determined at a specific time. It is possible for a value to be determined very precisely at two different dates, and for the values at each of these respective dates to differ. Although the ultimate stability of a point cannot be determined by a single observation session, an attempt should be made to select sites which are believed to be tectonically stable.

The classification of first-order, class i is reserved for network points that have demonstrated long-term stability.To ermsre this stability, the point should be rootwved at least twice during the year of establishment and thereafter at sufficient intervals. The long-term drift should indicate that the value will not change by more than the 20 pGal for at least 5 years. A point intended as fnst.xder, class I will initially be classified as first-order, class II until stability during the first year is demonstrated.

The new survey is required to tie at least two network points, which should have an order (and class) equivalent to or better than the intended order (and class) of the new survey. This is required to check the validity of existing network points as well as to ensure instrument calibration. Users are encouraged to exceed this minimal requirement. However, if one of the network stations is a firstorder, class I mark, then that station alone can satisfy the

minimum connecting requirement if the intended order of the new survey is less than first-order.

l=trinnentatioa

Regardless of the type of a relative gravimeter, the internal error is of primary concern.

Order	First	First	Second	Third
Class	I	II		
Minimum instrument internal error(ono.uigma).(.sGtl)	10	10	20	30

The instrument's internal accuracy may be determined by performing a relative survey over a calibration line (see below) and examining the standard deviation of a single reading. This determination should be performed after the instrument is calibrated using the latest calibration information. Thus the internal error is the measure of instrument uncertainty after all possible systematic error sources have been eliminated by calibration.

Calibradon Procedures

An instrument should be properly calibrated before a geodetic survey is performed. The most important calibration item is the determination of the mathematical model that relates dial units, voltage, or some other observable to milligals. This may consist only of a scale factor. In~ other cases the model may demonstrate nonlinearity or penodicity. Most manufacturers provide tables or scale factors with each instrument. Care must be taken to ensure the validity of these data over time.

When performing firstsder wcd~ this cahlmdon model should be determined by a combination of bench tents and field measurements The bench tests are specified by the manufacturer. A field calibration should be performed over existing control points of first-order, class I or II.

The entire usable gravimeter range interval should be sampled to ensure an uncertainty of less than 5 ~sGaL FGCC member agencies have established calibration lines for this specific purpose.

The response of an instrument to air pressure and temperature should be determined. The meter should be adjusted or calibrated for various pressures and temperatures so that the allowable uncertainty from these sources does not exceed the values in the table below.

The manufacturer's recommendations should be followed to ensure that all internal criteria, such as galvanometer sensitivity, long and cross level or tilt sensitivity, and reading line, arc within the manufacturer's allowable tolerances.

The response of an instrument due to local orientation should also be determined. Systematic differences may be due to an instrument's sensitivity to local magnetic variations. Manufacturers attempt to limit or negate such a response. However, if a meter displays a variation with

respect to orientation, then one must either have the instrument repaired by the manufacturer, or minimize the effect by fixing the orientation of the instrument throughout a survey.

Order	First	First	Second	Third
Class	I	II		
Necessary for user to determine calibration model	Yes	Yes	Yes	No
Allowable uncertainty of calibration model (,&Gal)	S	5	10	IS
Allowable uncertainty due to external air temperature				

changes (isGal)1 1 3 —

Masumum uncertainty due to external air pressure changes (j&Gal) 1 1 2 —

Allowable uncertainty due to other factors (jaGal)3 3 5 —

Field Procedures

A relative gravity survey is performed using a sequence of measurements known as a loop sequence. There are three common types: ladder, modified ladder, and line.

The ladder sequence begins and ends at the same network point, with the survey points being observed twice during the sequence: once in forward running and once in backward running. Of course, more than one network point may be present in a ladder sequence.

Order First First Second Third

Class I II

Minimum number of instruments

usedinnurvey 2 2 2 1

Recommended number of instrumentsusedinsurvey 3 3 2 1

Allowable loop sequence a a a.b a.b.c

Minimum number of readings at each Obuervation/imtrument	5	5	2t	I
Standard deviation of consecutive readings (unclamped) from mean sottoexceed(uGal)	2	2	5	—
Monitor external temperature and atrpressure	Yes	Yea	No	No
Standard deviation of temperature measuremenu(C)	0.1	0.1	—	—
Standard deviation of air pressure measurement(mbar)	I	I	—	—
Standard deviation of height of utstrumentabove~,int(mm)	1	1	5	10

(a—ladder) (b—esodilicd ladder). (c—line)

Although two readings are reqwrod, only one reading send be ruendcd. nirrocted for lunisolar attraction~

The modified ladder sequence also begins and ends at the same network point. However, not all the survey points are observed twice during the sequence. Again, more than one network point may be observed in the sequence.

The line sequence begins at a network point and ends at a different network point A survey point in a line sequence is usually observed only once.

One should always monitor the internal temperature of the instrument to ensure it does not fluctuate beyond the manufacturer's recommended limits. The time of each reading should be recorded to the nearest minute.

Office Procedures

	First	First	Second	Third
Order	I	II		

Rejection J.iuuu

Maximum standard error of a

gravity value (joGs!) 2020 SO 100

Total allowable instrument

uncertainty(joGaI)10 10 20 30

Model Umceetaintion

Uncertainty of atmnepheric mass

model (joGal) 0.5 0.5— —

Uncertainty of lunisola.r

attraction(goGal)1 1 5 S Uncertainty of Earth elastic and

plastic response to tidal

loading (~oGal) 2 2 5 —A least squares adjustment, constrained by the network

configuration and precision of established gravity control, will be checked for blunders by examining the nornialized residuals$_0$ The observation weights will be checked by inspecting the postadjustment estimate of the variance of unit weight Gravity standard errors computed by error propagation in a correctly weighted least squares adjustment will indicate the provisional accuracy classification.

A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models which account fort

instrument calibrations

I) conversion factors (linear and higher order)
2) thermal response (if necessary)
3) atmnepheric pressure response (if necessary)

instrument drift

I) static

2) dynamic

atmospheric mats attraction (if necessary)

Earth tides

I) lunisolar attraction

2) Earth elastic and plastic response (if usemeazy)

4. Information

Geodetic control data and cartographic information that pertain to the National Geodetic Control Networks are widely distributed by a component of the National Geodetic Survey, the National Geodetic Information Branch (NGIB). Users of this information include Federal, State, and local agencies, universities, private companies, and individuals. Data arc furnished in response to individual orders, or by an automatic mailing service (the mechanism whereby users who maintain active geodetic files automatically receive newly published data for specified areas). Electronic retrieval of data can be carried out directly from the NGS data base by a user.

Geodetic control data for the national networks are primarily published as standard quadrangles of 30~ in latitude by 30’ in longitude. However, in congested areas, the standard quadrangles are 15’ in latitude by 15 in longitude. In mont areas of Alaska, because of the sparseness of control, quadrangle units are l in latitude by 1 in longitude. Data are now available in these formats for all horizontal control and approximately 65 percent of the vertical control. The remaining 35 percent are presented in the old formats; i.e., State leveling lines and description booklets. Until the old format data have been convened to the standard quadrangle formats, the vertical control data in the unconverted areas will be available only by complete county coverage. Field data and recently adjusted projects with data in manuscript form are available from NGS upon special request. The National Geodetic Control Networks are cartographically depicted on approximately 850 different control diagrams. NGS provides other related geodetic information: e.g., geoid heights, deflections of the vertical, calibration base lines, gravity values, astronomic positions, horizontal and vertical data for crustal movement studies, satellite-derived positions, UTM coordinates, computer programs, geodetic calculator programs, and reference materials from the NGS data ba~

The NGIB receives data from all NOAA geodetic field operations and mark-recovery programs. In addition, xher

Federal, State, and local governments, and private organizations contribute survey data from their field operations. These are incorporated into the NGS data base. NOAA has entered into formal agreements with several Federal and State Government agencies whereby NGIB publishes, maintains, and distributes geodetic data received from these organizations. Guidelines and formats have been established to standardize the data for processing and inclusion into the NGS data base. These formats are available to organizations interested in participating in the transfer of their files to NOAA (appendix

C).

Upon completion of the geodetic data base management system, information generated from the data base will be automatically revised. A new data output format is being designed for both horizontal and vertical published control information. These formats, which were necessitated by the requirements of the new adjustments of the horizontal and vertical geodetic networks, will be more comprehensive than the present versions.

New micropublishing techniques are being introduced in the form of computer-generated microforms. Some geodetic data are available on magnetic tape, tnicrofilni, and microfiche. These services will be expanded as the automation system ic fully implemented. Charges for digital data are determined on the bssis of the individual requests, and reflect processing time, materials, and postage. The booklets Publications of the National Geodetic Survey and Products and Services of the National Geodetic Survey are available from NGIB.

For additional information, write:

Chief, National Geodetic Information

Branch, N/CG17

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A.1 Authority

The US. Department of Commerce’s National Oceanic and Atmospheric Administration (NOAA) is responsible for establishing and maintaining the basic national horizontal, vertical, and gravity geodetic control networks to meet the needs of the Nation. Within NOAA this task is assigned to the National Geodetic Survey, a Division of the Office of Charting and Geodetic Services within the National Ocean Service. This responsibility has evolved from legislation dating back to the Act of February 10, 1807 (2 Stat. 413, which created the first scientific Federal agency, known as the “Survey of the Coast.” Current authority is contained in United States Code, Title 33, USC 883a, as amended, and specifically defumi by Executive Directive, Bureau of the Budget (now the Office of Management and Budget) Circular No. A-l6, Revised (Bureau of the Budget 1967).

To coordinate national mapping, charting, and surveying activities, the Board of Surveys and Maps of the Federal Government was formed December 30, 1919, by Executive Order No. 3206. “Specifications for Horizontal and Vertical Control” were agreed upon by Federal surveying and mapping agencies and approved by the Board on May 9, 1933. When the Board was abolished March 10, 1942, its functions were transferred to the Bureau of the Budget, now the Office of Management and Budget, by Executive Order No. 9094. The basic survey specifications continued in effect. Bureau of the Budget Circular No. A-16, published January 16, 1953, and revised May 6, 1967 (Bureau of the Budget 1967), provides for the coordination of Federal surveying and mapping activities~ “Classification and Standards of Accuracy of Geodetic Control Surveys.” published March 1, 1957, replaced the 1933 specifications. Exhibit C to Circular A-16, dated October 10, 1958 (Bureau of the Budget 1958), established procedures for the required coordination of Federal geodetic and control surveys performed in accordance with the Bureau of the Budget classifications and standards.

The Federal Geodetic Control Committee (FGCC) ~s chartered December 11, 1968, and a Federal Ccx,rdinator

for Geodetic Control and Related Surveys was appointed April 4, 1969. The FGCC Circular No. 1, “Exchange of

Information.” dated October 16, 1972, prescribes reporting procedures for the committee (vice Exhibit C of Circular A-16) (Federal Geodetic Control Committee 1972). The Federal Coordinator for Geodetic Control and Related Surveys. Department of Commerce, is responsible for coordinating, planning, and executing national geodetic control surveys and related survey activities of Federal agencies, financed in whole or in part by Federal funds. The Executive Directive (Bureau of the Budget 1967: p. 2) states:

(1) The geodetic control needs of Government agencies and the public at large are met in the most expeditious and economical manner possible with available resources; and

(2) all surveying activities financed in whole or in part by Federal funds contribute to the National Networks of Geodetic Control when it is practicable and economical to do so.

The Federal Geodetic Control Committee assists and advises the Federal Coordinator for Geodetic Control and Related Surveys.

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The covariance expresses the mutual intcrrelation between the two random variables. (Unlikc the standard deviation, the square root of the covariancc. if u,, is positive, has no meaning and therefore is not used.) Another term, the *correlation coef]icient. p,,.* is defined as

[f-u ___

H j **IH** (228)

(TO' L\ ~ /

where a~ and v' are the standard deviations of *x* and *v.* respectively.

2.18

COVARIANCE, COFACTOR, AND WEIGHT MATRICES

The one-dimensional case contains one random variable, x, with mean or expectation ~s and a variance *c4.* The two-dimensional case has two random variables, *x* and *v.* with means ,x~ and p~ and variances *u*-~ and *o*-~, respectively, and covariance *o*~. These three parameters can be collected in a *square svm,netric* matrix, *X,* of order 2, and called the *varianceCo variance matrix* or simply the *covaria,:ce matrix,* It is constructed as

- *[a~~J* (2.29)

where the variances are along the main diagonal and the covariance is off the dia~onal. The concept of the covariance matrix can be ex!ended to the ntultidimensional case by consid

ering ~r random variables x_1 . *x,* ,, arid writing x,, j (2.30)

which is an n ***X*** n square symmetric nlatrix.

Often in practice, the variances and covariances are not known in absolute terms hut only W a scale factor. The scale factor, given the symbol o~, is termed the *reference Vcirianc'e,* although other names, such as *variance factor* and *variance cxsocicrtecl ~itIt weight writs,* also have been used. The square root u~. of ***(Ta,*** is called the *reference .vtuudcrnl deviation,* classically known as the *standard error of unit* weight. The relative variances and covariances, called *cofactors,* are given by

Cr *(7*

= — and *q,~* (231)

(To (To

Collecting the cofactors in a square symmetric matrix produces the *cofactor matrix,* **Q,** with the obvious relationship with covariance matrix

Q = —&~ (2.32)

'Jo

When **Q** is nonsingular, its inverse is called the *weight matrix* and designated W; thus.

W = = ~$-l (2.33)

If o~ is equal to I or, in other words, if the covariance matrix is known, the weight matrix becomes its inverse.

= **J,, QJ~**

ss here **J.~** is *iii* x it and called the Jacobian nlatrix, or the partial derivative ofy with respect to x. with the following elements (see Section B.lO, Appendix B):

Uv~itv~ Pm 1

or1 ~2 *Ox,,*

CMCM CM

J a~

ox,,, 0),,,

Ox,

Equation (2.33) should be carefully understood, particularly in view of the *classical* dctinition of weights as being inversely proportional to variances. This clearly is not true *tuiless all* car *aricutces are equal to 0,* which means that all the random variables are mutually uncorrelated. Only then would ~ (and **Q)** become a diagonal matrix and the weight of the random variable become equal to *cr*~ divided by its variance.

2.19 \ARIANCE-COVARIANCE PROPAGATION

In Section *2.15,* we introduced the concept of propagation of errors, where consideration was eiven to the case of uncorrelated random errors. In this section, we generalize it to both correlated quantities and multiple functions of such quantities. This general case of propa2ation can be expressed as follows.

Lety be a set (vector) of *m* quantities, each of which is a function of another set (vector) .v of a random variables. Given the covariance matrix ~,, (or the cofactor matrix **Q,,)** for the variables x, the covariance matrix ~ (or cofactor matrix **Q,)** for the new quantities y may be evaluated from

= **~** (2.34)

or

(2.35)

(2.36)

a-;

(2.37)

0

(2.38)

Equation (2.34) or *(2.35)* is quite general inasmuch as multiple functions in terms of several variables are considered and, more important, no restrictions are imposed on the structure of the given covariance matrix **!,,,.** Therefore, the given random variables in general could be of unequal precision and correlated, so that **~,,** would no longer be a diagonal matrix. From the general propagation relationships of Equation (2.34), several relationships could he obtained. First, consider the case of a single function s of several (a) variables *x,,*

that are *u,tcorreicmteci* and with variances 4, *o~*   *a~,* respectively. Equation (2.34) would become

o   lPv/a.v ~jL~./,h j

2.20

MATHEMATICAL MODEL FOR ADJUSTMENT

Of course, if the variables *x,* were correlated, **X,, itt** Equation (2.37) would not be a dia~o-nal matrix and Equation (2.38) would include cross-product terms in all combinations. In such a case, it would be unwise to write the expanded form in Equation (2.38) but instead work directly with the matrix form.

Note that Equations (2.34) through (2.38) are given in terms of variances and covariances of the distributions. However, because such parameters are rarely knosvn in practice, the equations apply equally using sample variances and covariances.

In Section 2. 13 we introduced the notion that measurements are obtained for some, but not all, of the elenients of a *nzat/tettmaticai model* describing the geometric or physical situation at hand. It is important that the reader appreciate the importance of the concept of the mathematical model as well as its adequacy for the surveying problent. As an example, suppose we are interested in the area of a rectangular tract of land (assutned reasonably small to use coniputations in a plane). To compute that area, we need the length *a* and the width *b.* The area will naturally be computed from *A* = *ab,* which is the “functional” model. Now, it appears that all we have to do is go out with a tape, measure the length and width, each once, and apply the two values into the formula to compute the area. This sounds simple enough until we begin taking a closer look at the problem and analyzing the factors involved in its solution.

In addition to the random and systematic errors in tape measurements as discussed in Section 2.2 (see also Chapter 4), let us also examine the adequacy of the niodel *A* = *oh.* This niodel is formulated on the basic assumption that the parcel of land is exactly rectangular. What if one, or more, of the four angles is checked and differences from the right-angle assumption are discovered? The simple model given no longer could be used; instead, another that more properly reflects the *geometric* conditions should be utilized.

As another example, suppose that we are interested in the *s/mope* of a plane triangle. All that is required for this operation is to measure two of its angles, and the shape of the triangle will be uttiqttely determined. However, if we were to decide, for safety's sake, to measure all three angles, any attempt to construct such a triangle will immediately show *i'tconsiste,mcies* among the three observed angles. In this case the model simply is that the sum of the three angles niust equal 18O~. If three observations are used in this model, **it** is highly unlikely that the sum will equal exactly l80~. Therefore, svhen *rcdwzda,zt* observations, or more observations than are absolutely necessary. are acquired, these observations will rarely fit the *model* exactly. Intuitively, and relying on our previous discussion, this results from something characteristic to the observations and makes them inconsistent in the case of redundancy. Of course, we first need to be sure of the adequacy of the model (it is a plane triangle and not spherical or spheroidal, for exaniple). Then, we need to express the quality of the nieasurements before we seek to *cucljust* the observations to fit the model. These concepts are elaborated on in the follosving sections.

2.21

FUNCTIONAL AND STOCHASTIC MODELS

We indicated previously that survey measurements are planned with a mathematical model in mind that describes the physical situation or set of events for which the survey is designed. This mathematical model is coniposed of two parts: a functional model amid a stochastic

ntodel. The *functiomtal model* is the more obvious part, because it usually describes the geometric or physical characteristics of the survey problem. Thus, the functional model for the example concerning the triangle discussed at the end of the preceding section *involves the deterntination of the shape* of a plane triangle through the measurement of interior angles. If three angles in a triangle are available, redundant measurements are present with respect to the functional model. This does not say anything about the pro-, perties of the measured angles. For example, in one case, each angle ntay be measured by the same observer, using the same instrument, applying the sanie nieasuring technique, and performing the measurements under very siniilar environmental conditions. In such a case, the three measured angles are said to be equally "reliable." But certainly in other cases the resulting measurements will not be of equal quality. In fact, as most practicing surveyors know from experience, measurements always are subject to unaccountable influences that result in variability when observations are repeated. Such statistical variations in the observations are important and must be taken into consideration when using the survey measurements to derive the required information. The *stoc/tastic model* is the part of the matheniatical model that describes the statistical properties of all the elements involved in the functional model. For example, in the case of the plane triangle, to say that each interior angle was measured and to give its value is insufficient. Additional information should also be included as to how well each angle was measured and if there is reason to believe in a statistical "correlation" (or interaction) among the angles, and if so, how much. The outcome of having a unique shape for the triangle from the redundant measurements depends both on knowing that the sum of its internal angles is l80~ (the functional model) and knowing the statistical properties of the three observed angles (the stochastic model).

2.22 LEAST-SQUARES ADJUSTMENT

The concept of adjustment was introduced in Section 2.13 and some simple adjustment methods in Section 2.14. A more systeniatic procedure of adjustment is least squares. which is most commonly used in surveying and geodesy. Most people refer to least squares as an adjustment technique equivalent to *estintatiott* in statistics. Although *adjustntemtt* is not the most precise term, it is appropriate because adjustment is needed when there are redundant observations (i.e., niore observations than are necessary to specify the niodel). In this case the observations given are not consistent with the model and are replaced by another set of estimates, classically called *adjusted observations* (which also is not a precise term), that satisfy the model. As an example, consider the determination of a distance between two points. The distance may be considered a random variable and, if measured once, would have one estimate, so that no adjustment is needed. On the other hand, if the distance were measured three times, there would likely be three slightly different values, .v_1, x_2. and x~. Since the model concerns a single distance that would be *uniquely* specified by one measurement, it is obvious that there are two *redwtdattt* measurements. It also is clear that the situation requires adjustment to have a unique solution. Otherwise, there are several different possibilities for the required distance. We can take any one ofx_1, x_2, x_3 or a combination of x_1 and .r~, or .r~ and x_3, or x_1 and x_3, or .r_1, x,, and .r_3. Therefore, having redundant observations makes it possible to have numerous ways of computing the desired values. The multiplicity of possibilities and arbitrarittess of choice ti obtaining the required informutation obviously is uttdesirable. lttstead, a process or technique must be found so that otte always would get one unique answer, which is derived front the data and is the “best~’ that can be obtained. This is why the relative confidence in, or merit of, the different observations should be taken into account when computing the best estimate, which is defined

as the estintate that deviates least from all the observations while considering their relative reliability. This is basically the role of least-squares adjustment.

As another example, consider the case of a plane triangle in which the three angles must add up to l80~. That the three angles must add to 1800 represents a functional relationship that reflects the geometrical system involved in the problem. If the shape and not the size of the triangle is of interest, it is unnecessary to observe the magnitudes of three angles, because two angles will be sufficient to determine the third from the functional relationship just ntentioned. However, in practice. the three attglcs a, fl. and y are tneasurcd whenever possible, and their sum likely will be different front 1 80g. Suppose that the sum of the angles exceeds 1800 by 3” of arc. Any two of the three measured angles would give the shape of the triangle, but all three possibilities, in general. will be different. Therefore, to satisfy the condition that the sum of the angles must be **1800,** the values of the observed angles must be altered. Here, there are numerous possibilities: 3” maybe subtracted from any one of the three angles, perhaps 2” could be subtracted from the largest angle. 1” from the second largest, and nothing from the third angle; or it ntay appear to be more satisfactory if 1” were subtracted from each angle, assuming ti’.at they are equally reliable, taken by the same instrument and observer under quite sintilar conditions. An alternative criterion to thosejust given may be to apply alterations that are proportionate to the relative magnitudes of the angles, or the magnitudes of their complements or supplements, or even inversely proportionate to such magnitudes. It is clear, then, that, although adjustment is necessary. the large number of possibilities given are quite arbitrary and a *criterio,m* is required in addition to the satisfaction of the functional model of summing the angle to 1800. Such is the *least-squares* criterion, which is introduced in the next section.

2.23 THE LEAST-SQUARES CRITERION

Let *I* designate the vector of given observations and *v* the vector of *residuals* (or alterations). svhich when added to *1* yields a set of new estintates, ***1.*** that is consistent with the ntodel:

$$\boldsymbol{I}=\boldsymbol{I}+\boldsymbol{v} \qquad (2.39)$$

The statistical or stochastic properties of the observations are expressed by either the covariance or cofactor matrix ~ or Q. respectively, or by the weight matrix XV. (Note that **XV** = **Q'** and XV — '~' if the reference variance ***oi~j*** is equal to unity.) With these variables, the general form of the least-squares criterion is given by

= —~ minimum (2.40)

Note that ~ is a scalar, for which a minimum is obtained by equating to 0 its partial derivative with respect to ***u.*** In Equation (2.40). the weight matrix of the observations **XV** is not necessarily a diagonal matrix, implying that the observations may be correlated. If the observations are uncorrelated, **XV** svill be a diagonal matrix and the criterion will simplify to

= ~ wc~ = $w_1$rj + w~c5 + **. . .** + w,,c~ —-~ minimum (2.41) which says that the sum of the weighted squares of the residuals is a minimum. Another, simpler case involves observations that are uncorrelated and of equal weight (precision). for which **XV** = **I** and ~ becomes

= t< = c~ + c~ + ... + c; —~ ntmnmmum (2.42)

2.24

REDUNDANCY AND THE MODEL

Before planning the acquisition phase of surveying data, a general model usually is specified either explicitly or implicitly. Such a ntodel is determined by a certain nuntber of variables and a possible set of relationships atttong thent. Whether or not an adjustment of the survey data is necessary depends on the amount of observational data acquired. A *minimum itioitber* of independent variables always is needed to determine the selected model uniquely. Such a minimum number is designated a_0. If it measurements are acquired (it > it)) with respect to the specified model, then the *redu,tdancv,* or (statistical) degrees of freedom, is specified as the amount by which *it* exceeds it_0. Denoting the redundancy by ***r,***

As illustratiotts, consider the following examples:

The case covered by Equation (2.42) is the oldest and may have accounted for the nante *least squares*, because it seeks the "least" sum of the squares of the residuals.

If we refer back to the example of measuring a distance three times and assume that *x*~, .V2, and x_3 are of equal precision (sveight) and uncorrelated, it can be shown that the ~ is a minimum if the best estimate.~ is taken as the arithmetic mean of the three observations. Similarly. if the three interior angles a, ~, and y in the plane triangle exantple have a unit wci~ht matrix, the method of least squares will yield all three residuals equal to — I Therefore. when each angle is reduced by I ". their suttt will he 1800 and the functional tttodcl will be satisfied. These two examples, as,ssell as several others, are worked out in detail in the follosving section.

$$\boldsymbol{r} = a — it_0 \qquad (2.43)$$

FIGURE 2.6

3

x

I. The shape of a plane triangle is uniquely determined by a minimum of two interior angles, or it~ = 2. If three interior angles are measured, then with it = 3. the redundancy is ***r*** = 1.

2.The size and shape of a plane triangle require a minimum of three observations, at least one of svhich is the length of one side; or it_0 = 3. If three interior angles and all three lengths are available, then with it = 6, the redundancy is ***r*** = 3.

3.In addition to the size and shape of the plane triangle, its location and orientation with respect to a specified Cartesian coordinate system .rv also are of interest (Figure 2.6). In this case, the minimum number of variables necessary to determine the model is it,~ = 6. which can be explained in one of two ways. From exantple 2. the size and shape requires that it_0 = 3. thcn the location of one point (e.g.. .r, and v~ in Figure 2.6) and the orientation of one side (e.g., a in the figure) add three more to make a minimum

2.25 CONDITION EQUATIONS—PARAMETERS

total of six. Another way to determine a_0 is to express the model as simply locating three points, (1, 2. 3 in the figure) in the two-dimensional coordinate system ***xy,*** which obviously requires *six* coordinates. If observations x_1, **Yt'** a are known in addition to the three interior angles and three sides, then with a = 9, the redundancy is ***r*** = 3.

The success of a survey adjustment depends to a large measure on the proper definition of the ntodel and the correct determination of ito. Next, the acquired measurements ntust relate to the specified model and have a set that is sufficient to determine the model. If this is not the case, the adjustment would not be meaningful. This can be illustrated by having three different measurements of *o,te* interior angle in a plane triangle. In this case, even though a = 3 and it_0 = 2, it is clear that the shape of the triangle cannot be determined front these data.

After the redundancy ***r*** is determined, the adjustment proceeds by writing equations that relate the model ~‘ariables to reflect the existing redundancy. Such equations will be referred to either as *cottclitio,t equations* or simply as *co,tditioos.* The number of conditions to be formulated for a given problem will depend on whether only observational variables are involved or other unknown variables as well. To illustrate this point, consider having two measurements a_1 and a, for the angle a. If no additional unknown variables are introduced, there will be only *one co,tdiuiott equatiott* corresponding to the one redundancy. or $a_1 — a_2 = 0$. Once the adjustment is performed, the least-squares estimate of the anele *a* is obtained from another relationship; namely, a = = a_1 + i.’$_1$ (or & = **&~).** Note that this relationship is almost self-evident. Nevertheless, such additional relations are required to evaluate other variables, as will be shown in the following example. As an alternative, a could be carried in the adjustment as an additional unknown variable. In such a case, one more condition trmust be written in addition to the one corresponding to ***r*** = 1 (i.e., there must be tsvo conditions). These may be written as

o — &~ = 0

a — a, = 0

c = ***r*** + *11* (2.44)

0 _ it _ *n*~ (2.45)

The additional unknown variable, which is a randont variable like the observations, will be called a *paranteter*. The one thing that distinguishes a parameter from an observation is that the parameter has no a priori sample value but the observation does. After the adjustment, both the observations and the parameters will have new least-squares estimates, as well as estimates for their eofactor or covariance matrices, as will be explained in later sections of this chapter.

To summarize, if the redundancy is ***r*,** there exist ***r*** independent condition equations, which can be written in terms of the given ii observations. If it additional unknown parameters are included in the adjustment, a total of

independent condition equations in terms of both the a observations and it parameters must be written. For the parameters to be functionally independent, their nuntber, it, should not exceed the minimum number of variables, itO. necessary to specify the model. Hence, the following relation must be satisfied:

Similarly, for the formulated condition equations to he independent, their nutttber *c* should he no lareer than the total number of observations, a. Hence.

$$r\_c\_,t \qquad (2.46)$$

TI) dctnonstrate these relations as well as elaborate further on the concept of a parameter. consider another example.

EX.ANIILE 2.8. Figure 2.7 is a sketch of a stttall level network that contains a hettch ttark (B.M.) and three points, the elevations of which are needed.

Solution. To determine these three elevations, five differences in elevation, *l~.* *1~.* are measured. The arrow along a line in Figure 2.7 (by convention) leads from a low point to a higher point.

The model involves the elevations of four points, of which one is a bench mark havine a known elevation. Therefore, the minimum number of variables needed to fully specify the model is n~ = 3. Given it = 5 observations, it follows that the redundancy is r = 2. Consequently, for the least-squares adjustntent, two independent condition equations are written in terms of the five observations. One possibility is to write one cottdition for each of the two loops. *a* and ***b.*** shown in Figure 2.7 as follows:

~)+ *12* — ***13*** = 0

/~+*l~/s* ~ (2.47)

After the adjustment, new estimates. *I*~. **/~.** for the tive observations are obtained. Frotn these new values the elevations of points. 1, 2. and 3 can be computed uniqLlely no ntatter which conibination of estimated observations is used. For instance, the elevation of point 2 ntay be coniputed in any one of the following ways attd its value would be identical:

elevation of point 2 = B.M. **+ ~ +** *L*

= B.M. + *1*~

= B.M. + *15* — ***14***

An alternative least-squares procedure is possible if the elevatiotts of points 1. 2. 3 are carried as parameters in the adjustment. In such a case. u = 3 and, according to Equatiott (2.44), the number of conditions would be c = 2 + 3 = 5. Hence, u takes on its upper limit of,t_0 = 3, attd likewise c is equal toit (see Equations (2.45) and (2.46)). Denoting the parameters by .v~, x_2, .v_3. the five condition equations may be written as

B.M. + *l*~ — *x*,= 0

.1~ + *1*~ —= 0

x=[a'~ o~]

The semimajor and semiminor axes of the corresponding ellipse are computed in the following manner. First, a second-degree polynotnia I (called the *cit (lraLteristic polvutootial)* is set up using the elements of ~ as

— *(o-~ + o~DA + (o~o~ — ok,)* = 0

The two roots, A_1 and A_2, of Equation (2.122) (which are called the *eigenvalues of~)* are computed, and their square roots are the semimajor and semiminor axes of the *stautdarcl* ***error ellipse,*** as shown in Figure 2.8. The orientation of the ellipse is determined by computing ***0*** between the .r axis and the semimajor axis from

tan 20 = 2cr,,

axes will be parallel to x and y. If the two coordinates are of equal precision or yr = the ellipse becomes a circle.

Consider the general case where the covariance matrix for the position of point P is given as

(2.121)

(2.122)

(2.123)

(I, = *x*-~ —

The quadrant of 20 is determined from the fact that the sign of sin 20 is the same as the sien of a-, and cos 20 has the same sign as *(o~ — o~).* Whereas, in the one-dimensional case, the probability of falling within +*o'* and —a' is 0.683, the probability of falling on or inside the standard error ellipse is 0.394. In a manner similar to constructing intervals with given probabilities for the one-dimensional case (Section 2.30), different size ellipses may be established, each with a given probability. It should be obvious that the larger the size of the error ellipse, the larger is the probability. Using the standard ellipse as a base. Table 2.4 (in the previous section) gives the scale multiplierk to enlarge the ellipse and the corresponding probability (see Mikhail 1979).

As an example. for an ellipse with axes a = 2.447a~ and $\boldsymbol{b}$ = 2.447b_1 where ***a,*** and ***b,*** are the semimajor and semiminor axes, respectively, of the standard ellipse, the probability that the point falls inside the ellipse is ***0.95.***

In surveying, one frequently is interested in the relative accuracy between two points. 1 and 2, in a horizontal network. Then, the coordinate differences are

1~

and the total covariance ntatrix for the coordinates is

Ri

Then,

d= = ~

and

~Jd =

from which

cr3, = — ~(r + ~.2

— I

cr3, *o*- —2cr,. , (2.124)

= LT,DI — a’,1, — *cT,,11+ 0,*

Introducing the variances. *o~,* and *a’s,,* and covariance. Q,,~. in Equations (2.122) and (2. 123), the elements of a relative error ellipse can be computed.

In the three-dimensional case, where the horizontal position as well as the elevation of the point is involved, the precision region becomes an ellipsoid. Table 2.4 (Section 2.30) gives the corresponding multipliers. For ntore details on error ellipsoids, the reader may consult Mikhail (1979).

The concepts of error ellipse and error ellipsoid are quite useful in establishing confidence regions about poittts determined by surveying techniques. These regions are measures of the reliability of the positional determination of such points. They could also he specihed in advance as a means of establishing specifications.

Although both absolute error ellipses (for points) and relative error ellipses (for lines) are used to evaluate adjustment quality, it is frequently more convenient to replace the two-dimensional representation by a one-dimensional single quantity (similar to a’ in Section 2.30). In this case, a circular probability distrihution is substituted for the elliptical probability distribution. Consequently, a single circular standard deviation, a’., is calculated from the two semiaxes of the error ellipse. The value of *o~* depends on the relative magnitudes of these axes.

Let

= [~, ~]

represent the covariance matrix for the x, y coordinates of a point. Then. A = $\boldsymbol{a}^2$ and = $\boldsymbol{b}^2$ are the eigenvalues (A_1 > A) of *s,,.* and ***a*** and ***b*** are the semintajor and semiminor axes of the error ellipse, respectively. Note that

tr(~,,) = *a~* + *o-~* = $A_1 + A_2 = a^2 + \boldsymbol{b}^2$

If *orn,* = ***b*** = x/~7 and *o,,~,* = *a* = **VX7,** then the value of the ratio a'm,n/a'mJ, determines the relationship used to calculate *a>*.

When *cr,,,/o-,,~,* is between 1.0 and 0.6,

0>= *(0.52220'rn,n* + 0.4778a'm~~)

(2.125)

(2.126)

(2.127)

A good approximation that yields a slightly larger *a>* (i.e., on the safe side) is given by

a> =0.5(a + ***b)***

which may be extended to the limit of Umin/a'm~~ ~ 0.2.

The ‘mean” accuracy measure. Th (see Equation 2. 120), is

$$a\text{-- —}(a\text{-}^2 + \quad \text{—}(a^2 + b^2) = \text{—}(A_1 + Al$$

2' I 2 2 -

and

= + a~)j

actually is applicable only' when O'rnn/a'mj~ is between 1.0 and 0.8, in which case it yields essentially the same value of *0*> as in Equation (2.126). As the ratio of a'mn/a'ma, decreases. a_1. from Equation (2.127) gets progressively' larger than that from Equation (2.126) with a maximum increase of about 20 percent at Umin/O'ma~ = 0.2.

Of course, the probability associated with the standard error circle is the same as for the standard error ellipse, 0.394. The multipliers given in Table 2.4 also still apply for circular errors of different probabilities. Figure 2.9 shows several standard error ellipses and their corresponding circles for several 0mm/a-max ratios.

FIGURE 2.9

Error ellipses and their corresponding error circles (from ***"ACIC Technical Report Na. 96,"*** United States Air Force. February 1962).

/

a,,,, a

(F **-**

FIGURE 2.10

Error ellipse for rectangular coordinates.

EXAMPLE 2.15. The position ofpointA in Figure 2.10 is determined by the radial distance ***r*** = 100 m, with *a'*, = ***0.5*** m. and the azimuth angle a = 60~, with *a'* = 0~30'. Compute the rectangular coordinates .r and ~ and the associated covariance matrix for point ***A.*** (Assume ***r*** and a to be uncorrelated.) Then calculate the semimajor and semiminor axes of the standard error ellipse and its orientation.

Solution

x = ***r*** sin a = 86.60

v = *r* c osaSO.00

[Ox bxl

Oa ***Or*** IF ***r*** cos a sin a _ 50 0.8661

= I ~ ~‘ IL **—*r*** sin a cos all F —86.6 ***0.5*** J

Loa *Orj*

The covariance matrix of the known variables. ***r*** and a. is a diagonal matrix because they are uncorrclated. It is equal to

= K = L^{00087} (0.5)2]

Applying Equation (2.34). the covariance matrix of the Cartesian coordinates is

[so 0.866 1~0.7569 x l0~ 0 iF 50 **—86.61**

= I II II I

L^{86_6} 0.5 1) 0.25 iL^{0866} 0.5

F0.3767—0.21951

= **I** **I**

L—0.2l950.6300j

This matrix expresses the reliability of the Cartesian coordinates of point ***A.*** According to Equation (2.122). the characteristic polvnontial is

A^2 — (0.3767 + 0.6300)A + (0.3767)(0.6300) — (~0.2l95)2 = 0 A^2 — l.0067A + 0.189141 = 0

from which the two roots (eigenvalues) are

A_1 = 0.7567; A~ = 0.25

y

x

x

types of instruments, errors in distance and direction can be propagated for a given procedure. These propagated errors then are compared with the allowable tolerance and the instrument or procedure can be modified if necessary. Various instrutnents.

procedures for use of these instruntents, attd error propagation tttethods are described in suhsequettt chapters. Considerable judgntcttt and practical experience itt sllrveyillg arc required in this phase of the survey design.

3.6

SELECTION OF COMPUTATIONAL PROCEDURES AND THE METHOD FOR DATA PRESENTATION

Systematic procedures for gatherittg. tiling. pruces~ing. and dissemittating of the survey data should be developed. Field notes and methods for recording data are described in Section 3.12.

The prevalence of modern, efficient electronic computing equipment virtually dictates that rigorous methods and adjustment techniques be utilized in all but the smallest of surveys.

Methods for filing or storing survey information should be standardized for all jobs and ought to allow current or future retrieval with no complications. The use of a Geographic or Land Information System (GIS or LIS) that has an interface with a Computer-Aided Drafting and Design (CADD) system (see Chapter 14) is highly recommended as an efficient way to exploit current electronic technology for storing and filing data.

The method for presenting the data must be carefully considered. For the example case. a conventional line map or an orthophoto map (see Chapter 14) with an overlay of contours would be appropriate for presentation to a planning commission. Selection of the method for presenting the data underscores the value of using GIS or LIS and CADD. For example, if a line map with an overlay of contours were chosen, then most surveying firms with GIS/CADD capability would be able to provide this product quickly at a minimum cost.

3.7

RELATION BETWEEN ANGLES AND DISTANCES

In Section 3.5 reference was made to the propagated errors in distance and in direction. These errors are used to determine the uncertainty in position for a point. Assume that estimated standard deviations in a distance ***r*** and direction a are *cx,* and *a',,* respectively. As developed in Section 2.3 1 and Example 2. ***15.*** the position of a point determined using ***r*** and a has an uncertainty region defined by an ellipse centered about a point located by ***r*** and a as shown in Figure 3.1. In Example 2.15, a', = ***0.5*** m, ***r*** = 100 m, a = **600,** and *a',* = 30, which produce an ellipse having a semiminor axis of *a',* = ***0.5*** m that is parallel to ***r.*** The semimajor axis of the ellipse = ***ra',*** = 0.87 m and is normal to line ***r.*** Note that *a',* = 0.61 m and *a',* = 0.79 m, both of which are less than the maximum uncertainty in the point = ***ro',,*** = 0.87 m.

Suppose that *a',,* is chosen so that *ro',,* = *a',* or cr,, = 0.5/100 = 0.00500 rad. In this case, the tsvo axes of the region of uncertainty are equal and the ellipse becomes a circle, as illustrated in Figure 3.2. Therefore, to have the same contribution from distance and angle errors, *a',,* should be about 0~l7'.

The preceding analysis illustrates the relationship between uncertainties in direction and distance and emphasizes the desirability of maintaining consistent accuracy in the two measurements. The error in distance is normally expressed as a relative precision of ratio

y

0 [

xis = *b* = 0.50 ni

x

FIGURE 3.1

Error ellipse for ***r*** = 100 *m*, **a** = 60~, a, = 0.50 m, and

a’,, = 30’.

x

FIGURE 3.2

Error ellipse for ***r*** = 100 m, a = 30~, and a’, = *ro’*$_0$.

of the error to the distance (see Section 2. 1 1). In the example, the relative precision is 0.5/100, or I part in 200. Similarly, the linear distance subtended by ***a’,,*** in a distance ***r*** equals 0.5 and the tangent or sine of the error or its value in radians is I part in 200. Accordingly, a consistent relation between accuracies in angles and distances will be maintained if the estimated standard deviation in direction equals *a’,/r* **radians or the** relative precision in the distance.

It is impossible to maintain an exact equality between these two relative accuracies; but with some exceptions, to be considered presently, surveys should be conducted so that the difference between aneular and distance accuracies is not great. Table 3. 1 shows, for various angular standard deviations, the corresponding relative precision and the linear errors for lengths of 1000 ft and 300 m. For a length other than 1000 ft or 300 m, the linear error is in direct proportion. A convenient relation to remember is that an angular error of 01’ corresponds to a linear error of about 0.3 ft in 1000 ft or 3 cm in 100 m.

To illustrate the use of the table, suppose that distances are to be measured with a **precision of** 1/10,000. **From the table the corresponding permissible** angular error is 20”. **As another example, suppose that the distance from the instrument to a desired point is** determined as 250 *m* with a standard deviation of 0.8 m. **For an angular error of** 10’ the

0.5m

x ___

TABLE 8.19

Point list for descriptions

Code	Description
	Hub with tack
TC	Top curb
FOB	Point of Beginning
BF	Barbed wire fence
PC	Property corner
RBC	Rebar with cap

in the stored file. For the collector shown in Figure 8.54, the code may consist of up to Seven alphabetic, numeric, or alphanumeric characters and the description is limited to 16 alphanumeric characters, punctuation, symbols, and Spaces.

8.38
RADIAL SURVEYS

A traverse pet-formed from a single station is called a *radial traverse.* The station utilized should be a coordinated point in the primary control network for the project and, for three-dimensional surveys, must have a known elevation. A second station is necessary for a backsight and a third known point is useful for checking. Although radial surveys can be performed using separate theodolite and EDNI or a modular total station system, a self-contained total station system definitely is preferable and the focus of the discussions that follow.

The method is best used where the terrain is open, with few restrictions to the line of sight. It may be employed to determine locations for traverse stations; fix the position of randomly positioned points, such as in topographic surveys (Chapter 15); or to set off the positions of precalculated points, as in construction surveys (Chapter 17).

Figure 8.56 shows a radial traverse executed from known control point ***A*** with a backsight on known point ***H.*** It is desired to locate the positions of traverse points ***B, C D, E,*** and ***F.*** The total station system is set over station ***A,*** the h.i. is measured and recorded, a backsight is taken on ***H,*** and horizontal and zenith angles and distances are observed to all stations ***B*** through ***E,*** with the number of repetitions being a function of the desired accuracy. The h.i.s for target sights also should be measured for all stations sighted. The procedure followed is similar to that developed for total station instruments in a three-dimensional survey, as described in Section 8.37. The measured data can be recorded manually, in a form similar to that shown in Figure ***8.53,*** or using a data collector. In the latter case, the process is initiated by using the pull-down menus of the data collector **software, and the sequence of operations with** the total station instrument is controlled by the ensuing prompts on the display screen of the collector. Most data collectors have routines that can be used for radial surveys.

Because lines *AH* and *AK* are of known directions, the azimuths of lines ***AB, AC, AD, AE, AA', and AF*** can be determined. Using the measured slope distances and zenith angles.

horizontal distances are computed, allowing the calculation of *X* and *Y* coordinates for all points by Equations (8.6) and (8.7). For three-dimensional surveys, the elevations are determined using the distances, zenith angles, and h.i.s at both ends of the line in Equation *(5.lla).*

o *K* 4

C

T

C

E

8 *A’*

H

A

FIGURE 8.56

Radial traverse.

To check the work, the entire operation is repeated from auxiliary station A’ with a backsight on station ***K,*** yielding a second set of azimuths, horizontal distances, and ~ elevations from ***A’*** to all stations ***A*** through ***F.*** These extra measurements provide a second set of coordinates and elevations for all traverse points. If the values for the two sets of positions are in reasonable agreement, the mean coordinates and elevations are used as the final position for the stations. Alternatively, data from both points ***A*** and ***A’*** can be used in a least-squares solution for the best estimates of the positions. With the adjusted positions. distances and directions between the coordinated stations can be calculated for lines *AB. BC. CD, DE,* ***EF,*** and FA using Equations (8.8) and (8.9). If distance A’H is observable, the trianele ***AA’H*** should be calculated as a check.

Note that the entire procedure could be performed with a backsight on a single line such as ***AH*** or ***AK.*** However, the extra known point provides an additional check on direction, and if the distance from A’ to ***K*** can be measured, a redundancy on the position of ***A’*** is furnished. In either case, an auxiliary station such as ***A’*** is absolutely necessary to provide a check on each point located.

To improve the final positions adopted for the stations, rigorous adjustment tnethods can be performed to reconcile redundant data. The linearized form of the distance, direction. and *L* elevation Equations (8.8), (8.9), and (5.1 Ia) can be formed for each ray from ***A*** and -1' to the traverse points. The system of equations so formed then could be solved by the method of least squares in a procedure similar to that developed for a three-dimensional traverse in Section 9.21.

It should be emphasized that there is no standard solution for the application of radial surveys to surveying problems. Each problem is unique and the basic rules as outlined must be used to suit the situation in the field.

8.39

TRAVERSE ADJUSTMENT BY LEAST SQUARES

As explained in Section 8.1, a traverse is composed of consecutive distance and angle measurements. Figure 8.57a shows a traverse between two horizontal control points. 1 and

5, at each of which the azimuths A_1 and A_5 also are known. The observations are five angles.

a_1 to a_5, and four distances, ***l~*, to *145.* A traverse can be adjusted using either of the** two

techniques of least squares presented in Chapter 2. Sections 2.27 and 2.28. The *technique*

Error sources in horizontal angle

Weather total station is used, errors are present in every horizontal angle measurement. Weather an instrument circles read, a small error is introduced in the final measured angle. Also, in pointing to a target, a small amount of error always occurs. Other major error sources in angle measurement include instrument and target setup error, and instrument leveling. Each of these sources products random errors. They may be small or large, depending on the instrument, the operator, and the conditions at the time of the angle measurement. Increasing number of angle can reduce the effect of reading, pointing, and leveling errors. However, increasing sight distance can only reduce the effect of the instrument and target setup errors.

Reading Errors:

Errors in reading conventional transits and theodolites are dependent on the quality of the instruments optics, the size of the smallest division of the scale, and the observer's abilities.

Typical reading errors for a 1" micrometer theodolite can range from tenths of a second to several seconds.

Reading errors also occur with digital instrument, their size being dependent on the sensitivity of the particular electronic angular resolution system.

Typical values range from ± 1" for the precise instrument to ± 10" for the less expensive ones. These errors are random, and effect on an angle.

Angles Measured by the Repetition Method:

When measuring a horizontal angle by repetition method, the first occurs when the circle is zeroed, and the second when reading the final cumulative angle.

$$\pm \sim_r = \frac{\pm r\sqrt{2}}{n}$$

Angles Measured by the Direction Method:

When a horizontal angle is measured by the directional method, the horizontal circle is read in both the back sight and foresight directions.

$$\pm\sim_r = \frac{\pm r\sqrt{2}}{\sqrt{n}}$$

$\pm\sim_r$: Estimated angular error due to when the directional method.

$\pm_r$: Observer reading error.

n: number of repetition.

Pointing errors

Accuracy in pointing to target is dependent on several factors. These include the optical qualities, target size, the observers personal, the weather conditions at time of observations.

$$\pm\sim_p = \frac{\pm p\sqrt{2}}{\sqrt{n}}$$

Where, $\pm\sim_p$ is the estimated contribution to the overall angular error due to pointing.

$\pm_p$ Estimed errors in pointing, n = number of angle measurements.

Target-centering errors

Whenever a target is set over a station, there will be some error due to faulty centering. It can be attributed to environmental conditions, optical plummet errors, and plumb bob centering error, personal abilities. The instrument usually within 0.000305 to 0.00305 m. $\pm\sim_t = \frac{\sqrt{D_1^2 + D_2^2}}{D_1 D_2}\pm_t\,°$

Where $\pm\sim_t$ is the angular error due to target, D_1, D_2 are the distance from target to the instrument at station 1,2, respectively. $° = 206264.8$ “.

Instrument centering errors

Every time an instrument is centered over a point, there is some error in its position with respect to the true value station location. This error dependent on the quality of the instrument, the optical plummet, the quality of tripod, and the skill of the observer.

$\pm\sim_i$ ■⌖ $\dfrac{D_3}{D_1 D_2 \sqrt{2}} \pm i^{\circ}$

Where $\pm\sim_i$ error due to faulty instrument centering.

$$D_3^2 = D_1^2 + D_2^2 - 2D_1 D_2 \cos\sim$$

Primary designed standards deviations for angles and distances measured on the maps.

Angles	$\sim_r$	$\sim_p$	$\sim_t$	$\sim_i$
$\sim_1$	⌖	⌖	⌖	⌖
$\sim_2$	⌖	⌖	⌖	⌖
$\sim_3$	⌖	⌖	⌖	⌖
$\sim_4$	⌖	⌖	⌖	⌖
$\sim_5$	⌖	⌖	⌖	⌖
$\sim_6$	⌖	⌖	⌖	⌖

Angles	±~	Distances	±d
$\sim_1$	⌖	1	⌖
$\sim_2$	⌖	2	⌖
$\sim_3$	⌖	3	⌖
$\sim_4$	⌖	4	⌖
$\sim_5$	⌖	5	⌖
$\sim_6$	⌖	6	⌖

$$\pm_{total} = \sqrt{(2.18)^2 + (2.151)^2 + (3.253)^2 + (3.302)^2 + (2.253)^2 + (4.889)^2} = ⌖7.736''$$

$$\pm_{expected} = ⌖7.736''$$

$$\pm_{expected} * C_t = \pm_{spesification}$$

C_t = 2.541 , (t-distribution cofactor)

$$7.736 * 2{,}541 = 19.657$$

$$\pm_{spe} = 12\sqrt{6} = 29.39$$

$$\pm_{expected} = \frac{\pm_{spe}}{C_t} = \frac{29.39}{2.541} = 11.567$$

our $\pm_{expected}$ computed is less than $\pm_{spe}$

obsevation	*unknowns*							
	dx_B	dy_B	dx_C	dy_C	dx_D	dy_D	dx_E	dy_E
AB	$\frac{X_B-X_A}{(AB)_\circ}$	$\frac{Y_B-Y_A}{(AB)_\circ}$	0	0	0	0	0	0
BC	$\frac{X_B-X_C}{(BC)_\circ}$	$\frac{Y_B-Y_C}{(BC)_\circ}$	$\frac{X_C-X_B}{(BC)_\circ}$	$\frac{Y_C-Y_B}{(BC)_\circ}$	0	0	0	0
CD	0	0	$\frac{X_C-X_D}{(CD)_\circ}$	$\frac{Y_C-Y_D}{(CD)_\circ}$	$\frac{X_D-X_C}{(CD)_\circ}$	$\frac{Y_D-Y_C}{(CD)_\circ}$	0	0
DE	0	0	0	0	$\frac{X_D-X_E}{(DE)_\circ}$	$\frac{Y_D-Y_E}{(DE)_\circ}$	$\frac{X_E-X_D}{(DE)_\circ}$	$\frac{Y_E-Y_D}{(DE)_\circ}$
EF	0	0	0	0	0	0	$\frac{X_E-X_F}{(EF)_\circ}$	$\frac{Y_E-Y_F}{(EF)_\circ}$
$-_1$	$\left(\frac{Y_B-Y_A}{(AB)^2_\circ}\right)*\circ$	$\left(\frac{X_A-X_B}{(AB)^2_\circ}\right)*\circ$	0	0	0	0	0	0
$-_2$	$\left(\frac{Y_A-Y_B}{(AB)^2_\circ}-\frac{Y_C-Y_B}{(BC)^2_\circ}\right)*\circ$	$\left(\frac{X_B-X_A}{(AB)^2_\circ}-\frac{X_B-X_C}{(BC)^2_\circ}\right)*\circ$	$\left(\frac{Y_C-Y_B}{(BC)^2_\circ}\right)*\circ$	$\left(\frac{X_B-X_C}{(BC)^2_\circ}\right)*\circ$	0	0	0	0
$-_3$	$\left(\frac{Y_C-Y_B}{(BC)^2_\circ}\right)*\circ$	$\left(\frac{X_B-X_C}{(BC)^2_\circ}\right)*\circ$	$\left(\frac{Y_B-Y_C}{(BC)^2_\circ}-\frac{Y_D-Y_C}{(CD)^2_\circ}\right)*\circ$	$\left(\frac{X_C-X_B}{(BC)^2_\circ}-\frac{X_C-X_D}{(CD)^2_\circ}\right)*\circ$	$\left(\frac{Y_D-Y_C}{(CD)^2_\circ}\right)*\circ$	$\left(\frac{X_C-X_D}{(CD)^2_\circ}\right)*\circ$	0	0
$-_4$	0	0	$\left(\frac{Y_D-Y_C}{(CD)^2_\circ}\right)*\circ$	$\left(\frac{X_C-X_D}{(CD)^2_\circ}\right)*\circ$	$\left(\frac{Y_C-Y_D}{(CD)^2_\circ}-\frac{Y_E-Y_D}{(DE)^2_\circ}\right)*\circ$	$\left(\frac{X_D-X_C}{(DC)^2_\circ}-\frac{X_D-X_E}{(DE)^2_\circ}\right)*\circ$	$\left(\frac{Y_E-Y_D}{(DE)^2_\circ}\right)*\circ$	$\left(\frac{X_D-X_E}{(DE)^2_\circ}\right)*\circ$
$-_5$	0	0	0	0	$\left(\frac{Y_E-Y_D}{(DE)^2_\circ}\right)*\circ$	$\left(\frac{X_D-X_E}{(DE)^2_\circ}\right)*\circ$	$\left(\frac{Y_D-Y_E}{(DE)^2_\circ}-\frac{Y_F-Y_E}{(EF)^2_\circ}\right)*\circ$	$\left(\frac{X_E-X_D}{(DE)^2_\circ}-\frac{X_E-X_F}{(EF)^2_\circ}\right)*\circ$
$-_6$	0	0	0	0	0	0	$\left(\frac{Y_F-Y_E}{(EF)^2_\circ}\right)*\circ$	$\left(\frac{X_E-X_E}{(EF)^2_\circ}\right)*\circ$

LIST OF FIGURES

CHAPTER 15 Control and Topographic Surveying

6.1GENERAL

By definition, a control survey consists of determining the horizontal and vertical or spatial positions of arbitrarily located points (Chapter 1, Section 1.6). The surveying methods for establishing control are described in Chapters *5* (vertical), 8 (horizontal by traversing), 9 (horizontal by intersection, resection, trilateration, and triangulation), and 12 (horizontal and vertical by the global positioning system). Traditionally, horizontal and vertical controls have been established separately, but with the advent of GPS, which provides both horizontal and vertical control in the same operation, this separation is rapidly disappearing.

A geodetic control survey, as described in Section 1.4. takes into account the shape of the earth and generally is used for primary control networks of large extent and high precision, such as those surveys established for continents, states, and counties. The bulk of the geodetic surveys performed currently are done with GPS for the horizontal positions but geodetic leveling (Section *5.52)* still is used for precise vertical control. By virtue of the characteristics of the system and the reduction process. differential GPS automatically yields a geodetic horizontal survey.

An engineering control survey provides the horizontal and vertical control for the design and construction of private and public works. Depending on the size and scope of the project, such a survey may be geodetic but often is simply a plane survey for horizontal control with precise or differential leveling for vertical control. Ideally, the engineering survey should originate and close on horizontal and vertical control points in the national or state geodetic network. Naturally, GPS surveying methods also are applicable to engineering surveys.

The distinguishing feature of a topographic survey is the determination of the location, both in plan and elevation, of selected ground points that are necessary for plotting contour lines and the planimetric location of features on the topographic map. A topographic survey consists of (1) establishing, over the area to be mapped. a system of horizontal and vertical controls, which consists of key stations connected by measurements of high precision; and

(2) locating the details, including selected ground points, by measurements of lower precision

from the control stations.

Topographic surveys fall roughly into three classes, according to the map scale employed as follows:

Large scale 1:1200(1 in. to 100 ft) or larger
Intermediate scale 1:1200 to 1:12,000 (1 in. to 100 ft to 1 in. to 1000 ft)
Small scale 1:12,000 (1 in. to 1000 ft) or smaller

Because of the range in uses of topographic maps and variations in the nature of the areas mapped, topographic surveys vary widely in character.

Topographic surveys can be performed by aerial photogrammetric methods, ground survey methods, or some combination of these two procedures. The largest portion of almost all of the small- and intermediate-scale as well as some large-scale topographic mapping now is performed by photogrammetric methods (Section 13.26). This photogrammetric operation includes establishing portions of the horizontal control in addition to compilation of the topographic map. However, ground survey methods still are applicable for large-scale topographic mapping of small areas and for field completion surveys, which usually are needed for photogrammetrically compiled topographic maps. The discussions in this chapter are directed primarily toward the various procedures for topographic surveys by ground survey methods.

15.2
PLANNING THE SURVEY

The choice of field methods for topographic surveying is governed by (1) the intended use of the map, (2) the area of the tract, (3) the map scale, and (4) the contour interval.

1. *Intended use of the map.* Surveys for detailed maps should be made by more refined methods than surveys for maps of a general character. For example, the earthwork estimates to be made from a topographic map by a landscape architect must be determined from a map that represents

the ground surface much more accurately in both the horizontal and vertical dimensions than one to be used in estimating the storage capacity of a reservoir. Also, a survey for a bridge site should be more detailed and more accurate in the immediate vicinity of the river crossing than in areas remote there from.

2. *Area of the tract.* It is more difficult to maintain a desired precision in the relative location of points over a large area than over a small area. Control measurements for a large area should be more precise than those for a small area.
3. *Scale of the map.* It sometimes is considered that, if the errors in the field measurements are no greater than the errors in plotting, the former are unimportant. But, because these errors may not compensate each other, the errors in the field measurements should be considerably less than the errors in plotting at the given scale. The ratio between field errors and plotting errors should be perhaps one to three.

   The ease with which precision may be increased in plotting, as compared with a corresponding increase in the precision of the field measurements, points to the desirability of reducing the total cost of a survey by giving proper attention to the excellence of the work of plotting points, interpolation, and interpretation in drawing the map.

   The choice of a suitable map scale is discussed in Section 14.7.
4. *Contour interval.* The smaller the contour interval, the more refined should be the field methods. The choice of a suitable contour interval is discussed in Section 14.7.

15.3
ESTABLISHMENT OF CONTROL

Control consists of two parts: (I) *horizontal control,* in which the planimetric positions of specific control points are located, and (2) *vertical control,* in which elevations are established on specified bench marks located throughout the area to be mapped. This control provides the skeleton, which later is clothed with the *details,* or locations of such objects as roads, houses, trees, streams, ground points of known elevation, and contours.

On surveys of wide extent, relatively few stations distributed over the tract are connected by more precise measurements forming the *primary control;* within this system, other control stations are located by less precise measurements, forming the *secondary control.* For small areas, only one control system is necessary, corresponding in precision to the secondary control used for large areas.

15.4
HORIZONTAL CONTROL

Horizontal control can be established by GPS survey, total station system traverse, aerial photogrammetric methods, ordinary traverse, or trilateration and triangulation. Frequently, a combination of certain of these methods is used.

GPS surveys using static (Section 12.8) and rapid static (Section 12.11) methods and total station system traverse (Section 8.37) can be used to establish primary and secondary control for relatively large topographic surveys. These methods also are utilized in areas of lesser extent when field conditions are appropriate (hilly, urban, or mountainous regions). Specifications and standards for these types of surveys can be found in Sections 8.21 and 8.41 for traverse, and in Federal Geodetic Control Committee (1989) for GPS surveys.

Horizontal control determination by aerial photogrammetric methods (Section 13.22) is feasible and particularly applicable to small-scale mapping of large areas. Note that traditional photogrammetric control surveys require a basic framework of horizontal control points established by GPS or total station traverse. However, if a GPS receiver is used aboard the aircraft procuring the aerial photography, the number of ground control points can be substantially reduced (Section 13.22), although as yet, not eliminated.

15.5
VERTICAL CONTROL

The purpose of vertical control is to establish bench marks at convenient intervals over the area to serve (1) as points of departure and closure for operations of topographic parties when locating details and (2) as reference marks during subsequent construction work.

Vertical control usually is accomplished by direct differential leveling (Section *5.33),* but for small areas or in rough country the vertical control is frequently established by trigonometric leveling (Sections *5.5, 5.55).*

All elevations for topographic mapping should be tied to bench marks that are referred to the North American Vertical Datum of 1988 (N.AVD 88; see Section 14.2).

Specifications for first-, second-, and third-order differential levels are given in Table *5.2,* Section 5.47 and Table *5.3,* Section *5.59.* These specifications may be relaxed somewhat depending on map scale, character of the terrain to be mapped, the contour interval desired, and ultimate use of the survey. Table 15.1 gives the ranges of approximate

TABLE 15.1

Topographic survey vertical control specifications

		Length of circuit **ml**	 **km**	**Maximum error of Closure** **ft**	 **mm**
Intermediate	Primary	1—20	2—30	0.05—0.3 xV~i	12—72 xV~
	Secondary	1—5	2—8	*0.1—0.5* xV~i	24—120 xV~
Large	Primary	*1—5*	2—8	0.05—0.1 *xV~i*	12—24 xVi~
	Secondary	;—3	*1—5*	*0.05—0.1* *xV~ii*	12—24 xV~

closures applicable to intermediate- and large-scale topographic mapping surveys. The smaller error of closure for a given map and type of control is used for very flat regions, where a contour interval of *0.5* m (1 ft) or less is required, and on surveys that are to be used to determine gradients of streams or to establish the grades of proposed drainage or irrigation systems. The higher errors of closure apply to surveys in which no more exact use is made of the results other than to determine the elevations of ground points for contours having *0.5*-, 2-, and 3-rn or 2-, *5*-, and 10-ft intervals.

Revised accuracy standards, such as those currently being developed by the Federal

Geographic Data Committee (FGDC, 1997; also see Section 14.26), may be adopted in the near future. New standards such as these, if adopted, would supersede the values given in ~ Table *15.1*. Consequently, a constant surveillance of the publications of the surveyor's professional organizations is required to stay abreast of developments in the continuing evolution of accuracy standards.

When an adequate number of points, having known elevations referred to the datum and a reliable mathematical function to model the geoid in the region, are available elevations by differential GPS survey may be used (Section 12.13, Part 5). Bear in mm that elevations so determined with current methods may have errors in the 3- to 5-cm range. Specifications and standards governing GPS elevations can be found in FGCC (1989). When accuracies required for elevations permit, GPS can provide elevations with substan-. tial improvement in speed of acquisition and economy.

- ***15.6***

HORIZONTAL AND VERTICAL CONTROL BY THREE-DIMENSIONAL TRAVERSE

A three-dimensional, total-station system traverse (Section 8.37) can be used for establishing control for intermediate- and large-scale topographic mapping jobs. Care should be exercised in the trigonometric leveling aspect, with changes in elevations being determined in both directions for each traverse line and corrections for earth curvature and refraction applied for all long lines (Section *5.55).*

Differential GPS surveys (Sections 12.8 and 12.9) automatically provide the th dimension. Such surveys are satisfactory for horizontal and vertical control establishment. for topographic surveys, assuming that the qualifications with respect to elevatiOnS, **SS** detailed in the previous section, are present.

Scale of **map**

Type of **control**

15.7

LOCATION OF DETAILS

In the following sections, the horizontal and vertical controls are assumed to have been established and the field party is concerned only with the location of details.

The adequacy with which the resulting map sheet meets the purposes of the survey depends largely on the task of locating the details. Therefore, the topographer should be informed as to the uses of the map so that proper emphasis is placed on each part of the work.

The instruments currently most used for the location of details are the total station system and GPS equipment. The engineer’s optical readin2 theodolite and engineer’s level with level rod or stadia rod, although still used on certain types of topographic surveys, rapidly are being displaced by the previously mentioned electronic systems.

The principal procedure for acquiring topographic detail in the field, using current equipment (e.g., total station system, GPS, or theodolite stadia), is the controlling point method. Other classical procedures, using theodolite, tape. rod, and the like, are the cross-profile, checkerboard, and trace-contour methods. The cross-profile or cross-section method, which is applied primarily in route surveying, is discussed in Chapter 16, Section 16.24. With the prevalence of total station systems and GPS equipment in current surveying operations, much less emphasis now is placed on the checkerboard and trace contour methods for topographic surveying. Therefore. in this chapter, the major focus is on use of the controlling point method for locating topographic details.

15.8
PRECISION

The precision required in locating such definite objects as buildings, bridges, curbs, inlets, and boundary lines should be consistent with the precision of plotting, which may be assumed to be a map distance of about *0.5* mm or ~ in. Such less definite objects as shorelines, streams, and the edges of wooded areas are located with a precision corresponding to a map distance of perhaps 0.9 to 1.3 mm or to ~ in. For use in maps of the same relative precision, more located points are required for a cven area on large-scale surveys than on intermediate surveys; hence, the location of details is relatively more important on large-scale surveys.

- Contours

The accuracy with which contour lines represent the terrain depends on (I) the accuracy and precision of the observations, (2) the number of observations, and (3) the distribution of the points located. Although ground points are definite, contour lines must necessarily be generalized to some extent. The error of field measurement in a plan should be consistent with the error in elevation, which in general should not exceed one-fifth of the horizontal distance between contours. The error in elevation should not exceed one-fifth of the vertical distance between contours. The purpose of a topographic survey will be better served by locating a greater number of points with less precision. within reasonable limits, than by locating fewer points with greater precision. Therefore. if for a given survey the contour Interval is 5 ft. a better map will be secured by locating with respect to each instrument station perhaps 50 points whose standard deviation in elevation is 1 ft than by locating 25 points whose standard deviation is only 0.5 ft. Similarly, if the contour interval is 2 m, it is better to have 50 points with a standard deviation of 0.4 m than 25 points with a standard deviation of 0.2 m.

A general principle that should serve as a guide in the selection of ground points may be noted. As an example, let it be supposed that a given survey is to provide a map that shall be accurate to the extent that if a number of well-distributed points is chosen at random on it, the average difference between the map elevations and ground elevations of identical points shall not exceed one-half of a contour interval (Section 14.26). Under this requirement, an attempt is made in the field to choose ground points such that a straight line between any two adjacent points in no case will pass above or below the ground by more than one contour interval. Therefore, in Figure *15.1,* if the ground points were taken only at *a, b. c, d,* and *e,* as shown, the resulting map would indicate the straight slopes *cd* and *de;* the consequent errors in elevation of *inn* and *op* on the profile amount to two contour intervals and show that additional readings should have been taken at the points *n* and *o.* The corresponding displacement of the contours on the map is shown by dashed and full lines in Figure 15.2.

Angles

The precision needed in the field measurements of angles to details may be readily deter-mined by relating it to the required precision of corresponding vertical and horizontal distances. For a sight at a distance of 300 m (1000 ft), a permissible error of 0.09 m (0.3 ft) ~ in elevation corresponds to a permissible error of 01’ in the vertical angle; likewise, a I permissible error of 0.09 m (0.3 ft) in azimuth (measured along the arc from the point

720

I

FIGURE 15.1

Effect of omission of significant ground points.

FIGURE 15.2 N

Errors in contour lines due to insufficient ground points.

sighted) corresponds to a permissible error of 01’ in the horizontal angle. Values for other lengths of sight or degrees of precision are obtained in a similar manner; therefore, if it is desired to locate a point to the nearest 2 ft in azimuth (or elevation) and if the length of the sight is 500 ft. the corresponding permissible error in the angle is 2/500 = 0.004 rad = 14’.

15.9 DETAILS BY THE CONTROLLING POINT METHOD

Details may be located with the controlling point method by employing a total station system (Sections 7.15—7.21) in radial surveys (Section 8.38). GPS kinematic surveys (Sections 12.9 and 12.10), and theodolite stadia (Sections 7.2—7.14). This method is applicable to practically every type of terrain and condition encountered in topographic mapping. Because only the controlling points that govern the configuration of the land and the location of planimetric features are located, a measure of economy is achieved not possible in the other techniques. However, the topographer must be experienced, because the success of the method depends on the selection of key controlling points. Procedures for use of this method are outlined in the following sections.

15.10

RADIAL SURVEYS BY THE TOTAL STATION SYSTEM

The personnel of the topographic party using a total station system usually consists of an instrument operator and one or two people operating reflectors mounted on prism poles.

The procedure is as follows. First, consider the case where the data are recorded manually in a form similar to that shown in Figure *15.3.* The total station instrument is set up on either a primary or secondary control station, the elevation and position of which are known. The height of instrument (h.i.) is measured with a rod, tape, or observed on the plumbing rod if the tripod is so equipped. The temperature and atmospheric pressure also are measured and recorded. The total station instrument power is switched on, the environmental data and reflector constant are entered, and a backsight is taken on another station in the primary or secondary network of control. The horizontal circle may be set either on

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124 1.950 S2240'O~ 04722' 90.45 90.20~. *720.21 0* titer *re* *il* _____

FIGURE 15.3

2.3 Gravity Control Network Standards

When a gravity amtrvl point is classified with a particular order and class, NGS certifies that the gravity value at that control point possesses a specific accuracy.

Gravity is commonly expressed in units of milligals (mGal) or microgais (sGal) equal, respectively, to (10-’) meters/sec’, and (10”) meters/sec^2. Classification order refers to measurement accuracies and class to site stability.

Table 2.3—Gravity accuracy standards

CIass~ficwiost	*Gratdry accuracy (~Ga1)*
Thst-order, class!	20 (subjcct tp stability verification)
First-order, class II	20
Second-order	50
Third-aider	100

When a survey establishes only new points, and where only absolute measurements are observed, then each survey point is classified independently. The standard deviation from the mean of measurements observed at that point is corrected by the error budget for noise sources ill accordance with the following formula~

c2~I(xIxfl,~~+c2 /

+*i* n—i

where

c —gravity accuracy

x—gravity measurement

n —number of measurements

xm — (2x)/n /

c—external random error

The value obtained for c is then compared directly against the gravity accuracy standards table.

When a survey establishes points at which both absolute and relative measurements are made, the absolute determination ordinarily takes precedence and the point is classified accordingly. (However, see Example D below for an exception.)

When a survey establishes points where only relative measurements are observed, and where the survey is tied to the National Geodetic Gravity Network, then the gravity accuracy is identified with the propagated gravity standard deviation from a minimally constrained, correctly weighted, least squares adjustment.

The worst gravity accuracy of all the points in the survey is taken as the provisional accuracy. If the provisional accuracy exceeds the gravity accuracy limit set for the intended survey classification, then the survey is classified using the provisional accuracy.

As a test for systematic errors, the variance factor ratio of the new survey is computed by the Iterated Almost Unbiased Estimator (IAUE) method described in appendix B. This computation combines the new survey measurements with existing network data which are assumed to be correctly weighted and free of systematic error. If the variance factor ratio is substantially greater than unity, then the survey does not check with the network, and both the survey and the network data will be examined by NGS.

Computer simulations performed by NGS have shown that a variance factor ratio greater than *1.5* typically indicates systematic errors between the survey and the network. Setting a cutoff value higher than this could allow undetected systematic error to propagate into the national network. On the other hand, a higher cutoff value might be considered if the survey has only a minimal number of connections to the network, because this circumstance would tend to increase the variance factor ratio.

In some situations, a survey has been designed in which different sections provide different orders of control. For these multi-order surveys, the computed gravity accuracies should be grouped into sets appropriate to the different parts of the survey. Then, the largest value of c in each set is used to classify the control points of that portion, as discussed above. If there are sufficient connections to the network, several variance factor ratios, one for each part of the survey, should be computed.

Gravity Examples

Example .4. Suppose a gravity survey using absolute measurement techniques has been performed. These points are then unrelated. Consider one of these survey points.

Assume n — 750

750

— Xm)^2 — .169 mGaP

e — *5* ~Gal

— 0.169 + *(.005Y*

750—1

c — l6MGal

The point is then classified as first-order, class II.

Example B. Suppose a relative gravity survey with an intended accuracy of second-order (50 pGaI) has been performed. A series of propagated gravity accuracies from a minimally constrained adjustment is now computed.

Station	*Gravity detzatian (p&Gal)*	*stardard*
1	38	
2	44	
3	—55	

Suppose that the worst gravity accuracy was *55* ~Gal. This is worse than the intended accuracy of *50* ~Gal.

Therefore, the provisional accuracy of *55* pGal would have precedence for classification, which would be set to third-order.

Now assume that a solution combining survey and network data has been obtained (as per appendix B) and that a variance factor of 1.2 was computed for the survey. This would be reasonably close to unity, and would indicate that the survey checks with the network. The survey would then be classified as third-order using the provisional accuracy of *55* ~Gal.

However, if a variance factor of, say, 1.9 was computed, the survey would not check with the network. Both the survey and network measurements then would have to be scrutinized to find the problem.

Example C. Suppose a survey consisting of both absolute and relative measurements hasbeen made at the same points. Assume the absolute observation at one of the points yielded a classification of rust-order, class II, whereas the relative measurements produced a value to second-order standards. The point in question would be classified as rust-order, class II, in accordance with the absolute observation.

Example D. Suppose we have a survey similar to Case C, where the absolute measurements at a particular point yielded a third-order classification due to an unusually noisy observation session, but the relative measurements still satisfied the second-order standard. The point in question would be classified as second-order, in accordance with the relative measurements.

Monumentation

Control points should be part of the National Geodetic Gravity Network• only if they possess permanence, horizontal and vertical stability with respect to the Earth's crust, and a horizontal and vertical location which can be defined as a point. For all orders of accuracy, the mark should be imbedded in a stable platform such as flat, horizontal concrete. For first-order, class I stations, the platform should be imbedded in stable, bard rock, and

checked at least twice for the first year to ensure stability. For first-order, class II stations, the platform should be located in an extremely stable environment, such as the concrete floor of a mature structure. For second and third-avier t.at~ons~ standard bench mark monumentation is adequate. Replacement of a temporary mark by a more permanent mark is not acceptable unless the two marks are connected in timely fashion by survey observations of

sufficient accuracy. Detailed information is given in *NOA~4 Manual NOS NGS* 1, "Geodetic bench marks." Monuments should not be near sources of electromagnetic interference.

It is recommended, but not necessary, to monument third-order stations. However, the location associated with the gravity value should be recoverable, based upon the station description.

Manually recorded note form, topographic details using total station system.

0.00055	0.00068	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
-0.00141	-0.00051	0.00086	-0.00017	0.00000	0.00000	0.00000	0.00000
0.00086	-0.00017	-0.00149	0.00025	0.00063	-0.00008	0.00000	0.00000
0.00000	0.00000	0.00063	-0.00008	-0.00155	0.00023	0.00092	-0.00015
0.00000	0.00000	0.00000	0.00000	0.00092	-0.00015	-0.00150	-0.00006
0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00059	0.00021
-0.77824	0.62796	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
-0.19779	-0.98024	0.19779	0.98024	0.00000	0.00000	0.00000	0.00000
0.00000	0.00000	-0.11826	-0.99298	0.11826	0.99298	0.00000	0.00000
0.00000	0.00000	0.00000	0.00000	-0.16207	-0.98678	0.16207	0.98678
0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.33961	-0.94057

Table5.1: Distance observations

FROM	TO	DISTANCE (m)	COMPUTED STANDARD DEVIATION (m)
M	A	1020.372	0.00
A	B	1135.444	0.84
B	C	1144.608	0.85
C	D	1574.147	0.76
D	E	1075.464	0.79
E	F	1604.423	0.78
F	N	1356.079	0.00

Table5.2: Angle observations

BACK SIGHT	OCCUPIED	FORESIGHT	ANGLE ° ' "	COMPUTED STANDARD DEVIATION
M	A	B	44 13 33.51	2.21
A	B	C	242 30 31	2.21
B	C	D	175 23 05.05	2.18
C	D	E	182 32 10.5	2.19
D	E	F	150 49 14.3	2.19
E	F	N	312 28 55.2	2.13

Table5.3: Control station

STATION	X (M)	Y (M)
M	107256.16	158950.33
A	107161.35	157933.10
F	101189.49	158775.68
N	101711.63	157523.07

Table5.4: Azimuth observations

FROM	TO	AZIMUTH ° ' "	COMPUTED STANDARD DEVIATION
1	2	264 40 30.6	0.00
3	4	292 38 21.61	5.3388

Table 5.5 Primary designed standards deviations for angles and distances measured on the maps.

ANGLES	~r	~p	~t	~i
~1	± 1.44"	± 1.44"	±0.829"	± 0.348 "
~2	±1.44"	±1.44"	± 0.78"	± 0.328"
~3	±1.44"	±1.44"	± 0.68"	±0.33"
~4	±1.44"	±1.44"	± 0.71"	± 0.342"
~5	±1.44"	±1.44"	± 0.708"	±0.33"
~6	±1.44"	± 1.44"	± 0.61	±0.12"

Table 5.6: estimated error in each angle and distance

ANGLES	±~	DISTANCES	±d
~1	± 2.209	AB	± 0.84
~2	± 2.21	BC	± 0.85
~3	± 2.177	CD	± 0.755
~4	± 2.188	DE	± 0.79
~5	± 2.186	EF	± 0.78
~6	± 2.133	FN	± 0.62

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SPATIAL DATA QUALITY

All data sources and spatial data entry methods present errors into the information that is created and used for display and analysis. The type, severity, and implications of these errors inherent in a Geographic Information System (GIS) database determine the quality of spatial data. These errors must be recognized and properly dealt with. It is virtually impossible to eliminate spatial data errors altogether, but GIS users can reduce and manage errors effectively, thus improving the quality of data.

Identifying and assessing data errors are not the only factors which determine data quality. Data quality includes all of the processes involved with developing, utilizing and maintaining a spatial database. These include data collection, data input, positional and attribute accuracy, data storage, data manipulation, data conversion, and quality control procedures. Identifying the history of a GIS database allows the user to determine the usefulness of the data in addition to data quality. Documentation about a spatial data set should include data sources, data input techniques, positional accuracy, attribute classifications and definitions, and quality control procedures used to validate the spatial data.

SIGNIFICANCE OF DATA QUALITY

Data sharing and data integration is an inevitable consequence of the widespread availability of powerful GIS technologies due to the high initial cost of establishing a spatial database. Because spatial data is transferred and shared by many users, the data must be trustworthy and useful. To ensure that existing digital data be appropriately used, the data producer must provide documentation about the history of spatial data. In addition to spatial data documentation, data developers and users have begun to document and implement data quality measurements. Data quality measures allow judgement to be made about spatial data. Spatial data is frequently relied upon as factual data and good data quality measures and documentation may eliminate liability law suits against data developers and data users. Data creators must be aware of the implications involved with carelessly developing spatial data if that data is intended to be used for lawful reasons. On the other hand, the data user must also be responsible for understanding the limitations of that spatial data.

DATA QUALITY VERIFICATION

Verifying the accuracy of spatial data is essential to data sharing and integration. Graphic and non-graphic (tabular) data need to be verified in order to ensure data quality in a GIS database. Data quality measurements verify positional accuracy, completeness, correctness, and integrity. Positional accuracy is defined by how well the true measurements of an object on the earth's surface match the same object stored as series of digital coordinates in a GIS data layer. Completeness measures the amount of spatial features included in a digital data set as a result of data input and conversion. Correctness describes how well the digital features match the objects on the earth's ground. For instance, is the road represented digitally truly a road or is it a stream? Lastly, integrity is another measurement of data quality specifically concerned with the completeness of relationships among data elements. Most procedural checks require manual validation which verifies accuracy and completeness of a spatial database. However, there is software available designed to automatically verify the integrity of a GIS database (Montgomery & Schuch, 1993).

Manual verification procedures include creating check plots, field checks and measurements. Check plots involve plotting the converted digital data at the same scale of the base map used to convert analog spatial data to digital spatial data. A base map contains spatial features and boundaries represented accurately and referenced to a specific coordinate system on a hard-copy map.

Once the digital map has been plotted, overlay it with the base map and measure the accuracy of four or more control points used to register the map. This will help determine whether the scale is true or off balance. Proceed to check the matching and tolerances of spatial features. Additionally, annotation and feature symbology are verified. Check plots are relatively inexpensive method for verifying spatial data. However, field and measurement checks are time consuming and expensive although these checks may yield more accurate measurements. In the field, feature locations are verified with a check plot along with identification of features through field surveying. A Global Positioning System (GPS) can also be used in the field to determine exact locations. A GPS is a highly accurate technology that uses earth-orbiting satellites to determine geographic location.

Automated data quality measurements search for logical inconsistencies and missing or strange attribute values. For instance, attribute values must contain acceptable or default range values. Inconsistent feature types within a data layer are also identified. For example, a data layer of soils polygons must contain enclosed polygons (Montgomery & Schuch, 1993).

Regardless of which method is employed, the user should be aware of the classification and attributes assigned to spatial data by the data developer. It is difficult to get a clear consensus from the GIG community for developing standards to classify and attribute spatial data. This of course is dependent upon the users needs. Some users may want data classified generally while others may be looking to obtain more specific data attributes for their needs.

Keep in mind that a GIS allows the user to verify features and coordinates about geographic entities by using a mouse to click on an point, area or line while the geographic extent is displayed on a graphics monitor. Although this is a quick way to verify individual data entities, the check plot allows you to verify all types of data at once. Ultimately the accuracy of spatial data is only as good as the data sources used to develop that spatial data.

DATA QUALITY MAINTENANCE

It is pertinent that data quality be maintained at all stages of a GIS database especially during data development and data maintenance. Any type of manipulation done on data affects the quality of that data so caution should be taken when adding, editing and updating spatial data. Errors induced at these stages will change the outcome of spatial data analysis. This can undermine the whole purpose and functionality of a GIS. Data developers should have documented rules and guidelines to follow when creating and updating data layers. This documentation helps to eliminate any future questions concerning data creation or data analysis and avoids any duplicate efforts made of creating specific data layers. Not only is this information useful for in-house data development, but data customers and users are able to determine the validity of data by checking the sources and procedures used to create the data.

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Table B-0: critical values for t distribution

~	0.400	0.350	0.300	0.250	0.200	0.150	0.100	0.050	0.025	0.010	0.005	0.001	0.0005
3													
1	0.325	0.510	0.727	1.000	1.376	1.963	3.078	6.314	12.70	31.82	63.64	318.2	636.2
2	0.289	0.445	0.617	0.816	1.061	1.386	1.886	2.920	4.303	6.964	9.925	22.40	31.58
3	0.277	0.424	0.584	0.765	0.978	1.250	1.638	2.353	3.183	4.541	5.842	10.22	12.95
4	0.271	0.414	0.569	0.741	0.941	1.190	1.533	2.132	2.776	3.748	4.604	6.897	8.610
5	0.267	0.408	0.559	0.727	0.920	1.156	1.476	2.015	2.571	3.365	4.032	5.895	6.880
6	0.265	0.404	0.533	0.718	0.906	1.134	1.440	1.943	2.447	3.143	3.708	5.208	5.961
7	0.263	0.402	0.549	0.711	0.896	1.119	1.415	1.895	2.365	2.998	3.500	4.785	5.408
8	0.262	0.399	0.546	0.706	0.889	1.108	1.397	1.860	2.306	2.896	3.356	4.510	5.041
9	0.261	0.398	0.543	0.703	0.833	1.100	1.383	1.833	2.262	2.821	3.250	4.304	4.781
10	0.260	0.397	0.542	0.700	0.879	1.093	1.372	1.812	2.228	2.764	3.169	4.149	4.605
11	0.260	0.396	0.540	0.697	0.876	1.088	1.363	1.796	2.201	2.718	3.106	4.029	4.452
12	0.259	0.395	0.539	0.695	0.873	1.083	1.356	1.782	2.179	2.681	3.055	3.933	4.329
13	0.259	0.394	0.538	0.694	0.870	1.079	1.350	1.771	2.160	2.650	3.012	3.854	4.230
14	0.258	0.393	0.537	0.692	0.868	1.076	1.345	1.761	2.145	2.624	2.977	3.789	4.148
15	0.258	0.393	0.536	0.691	0.866	1.074	1.341	1.753	1.131	2.602	2.947	3.734	4.079
16	0.258	0.392	0.535	0.690	0.865	1.071	1.337	1.746	2.120	2.583	2.921	3.688	4.021
17	0.257	0.392	0.534	0.689	0.863	1.069	1.333	1.740	2.110	2.567	2.898	3.647	3.970
18	0.257	0.392	0.534	0.688	0.862	1.067	1.330	1.734	2.101	2.552	2.878	3.611	3.926
19	0.257	0.391	0.533	0.688	0.861	1.066	1.328	1.729	2.093	2.539	2.861	3.580	3.887
20	0.257	0.391	0.533	0.687	0.860	1.064	1.325	1.725	2.086	2.528	2.845	3.553	3.853
21	0.257	0.391	0.532	0.686	0.859	1.063	1.323	1.721	2.080	2.518	2.831	3.528	3.822
22	0.256	0.390	0.532	0.686	0.858	1.061	1.321	1.717	2.074	2.508	2.819	3.506	3.795
23	0.256	0.390	0.532	0.685	0.858	1.060	1.319	1.714	2.069	2.500	2.807	3.486	3.770
24	0.256	0.390	0.531	0.685	0.857	1.059	1.318	1.711	2.064	2.492	2.797	3.467	3.748
25	0.256	0.390	0.531	0.684	0.856	1.058	1.316	1.708	2.060	2.485	2.787	3.451	3.727
26	0.256	0.390	0.531	0.684	0.856	1.058	1.315	1.706	2.056	2.479	2.779	3.435	3.708
27	0.256	0.389	0.531	0.684	0.855	1.057	1.314	1.703	2.052	2.473	2.771	3.421	3.691
28	0.256	0.389	0.530	0.683	0.855	1.056	1.313	1.701	2.048	2.467	2.763	3.409	3.675
29	0.256	0.389	0.530	0.683	0.854	1.055	1.311	1.699	2.045	2.462	2.756	3.397	3.661
30	0.256	0.389	0.530	0.683	0.854	1.055	1.310	1.697	2.042	2.457	2.750	3.385	3.647
35	0.255	0.388	0.529	0.682	0.852	1.052	1.306	1.690	2.030	2.438	2.724	3.340	3.592
40	0.255	0.388	0.529	0.681	0.851	1.050	1.303	1.684	2.021	2.423	2.704	3.307	3.552
60	0.254	0.387	0.527	0.679	0.848	1.045	1.296	1.671	2.000	2.390	2.660	3.232	3.461
120	0.254	0.386	0.526	0.677	0.845	1.041	1.289	1.658	1.980	2.358	2.617	3.160	3.374
∞	0.253	0.385	0.525	0.675	0.842	1.037	1.282	1.645	1.960	2.236	2.576	3.291	3.300

Tested for accuracy of map

Atopographic map can be tested for accuracy ,both in positional and in elevation.In this discussion it is assumed that the error in field measurment may be disregarded and that a graphical scale is provided on the map to render negligible any effect of shrinkage of the paper.

1. **Horizantal dimensions :** The test for horizantal dimension consits of comparing distances scaled from the map to distances measured on the ground between corresponding points.
2. **Elevation :** one test of the elevations consists of comparing, for selected points, the elevation determined by filed level and the corresponding taken from the map.

Test the map

- Determine data quality
- Collect the data about the map standard in Palestine from the palestinan community , and international map seandard .
- Select the map
- Select the stady area
- Select the important point on the map , and the same points in the study area , number of the points greater than **30** .
- We choose open travers to find the position of the important point in the field.
- From the information of the instrument (total station in P.P.U) , we find σd, $\sigma\alpha$ (standard error) .
- We use least square solution to determination the coordinat (X,Y) and the elevation Z.
- By use (L.S.S) we find $\sigma x, \sigma y$ for each point in the traverse .
- We compute error ellipses σc .

If σmin =b and σmax=a ,than the value of the ratio $\frac{\sigma min}{\sigma max}$ determine the relationship used to calculate σc.

When $0.6 \le \frac{\sigma min}{\sigma max} \le 1.0$, $\sigma c := (0.5222\,\sigma min + 0.4778\,\sigma max)$.

A good approximation that yield a slightly larger σc is given by :
$\sigma c := 0.5 \cdot (a + b)$.

where a = semi major axes , b = semi miner axes

Which may be extended to the limit of $\frac{\sigma min}{\sigma max} \le 0.2$ .The "mean" accuracy measure is

$$\sigma c := \left[\frac{1}{2}\cdot(\sigma x^2 + \sigma y^2)^2\right]^{\frac{1}{2}}$$

- After we compute $\sigma x, \sigma y$, σc from (L.S.S) and error ellipses ,we determine the order of the surveying work according to the level of the (FGCS) ,1st ,second , third .

- We find the mean absolute error $\vec{\delta}$ for x,y,z .

$$\overrightarrow{|\delta x|} := \left[\frac{\left[\sum_{i=1}^{n} (\delta xi)\right]}{n}\right]$$

$$\overrightarrow{|\delta y|} := \left[\frac{\left[\sum_{i=1}^{n} (\delta yi)\right]}{n}\right]$$

$$\overrightarrow{|\delta z|} := \left[\frac{\left[\sum_{i=1}^{n} (\delta zi)\right]}{n}\right]$$

Where $\delta x, \delta y, \delta z$ =discrepancies between the position from the map and the position determined by mean of an independent survey.

$\vec{\delta x}, \vec{\delta y}, \vec{\delta z}$ = mean algebraic deviations in the X,Y,Z components , (always +)

n = number of observed discrepancies ($n \ge 30$).

- Compute the standard error σ for each component as the standard error from the mean error.

$$\sigma x := \left[\frac{\sum_{i=1}^{n} \left[\left(\delta x - \overrightarrow{\delta x} \right) i \right]^2}{n - 1} \right]^{\frac{1}{2}}$$

$$\sigma y := \left[\frac{\sum_{i=1}^{n} \left[\left(\delta y - \overrightarrow{\delta y} \right) i \right]^2}{n - 1} \right]^{\frac{1}{2}}$$

$$\sigma z := \left[\frac{\sum_{i=1}^{n} \left[\left(\delta z - \overrightarrow{\delta z} \right) i \right]^2}{n - 1} \right]^{\frac{1}{2}}$$

- Point select for testing vertical accuracy do not have to be the same as those chosen for testing the horizontal accuracy.
- Point should be located as to be separated by **min 1/12, max 1/4** of diagonal dimension of the area.
- Comparing between the value of (X,Y,Z) in the field surveying and the value of (X,Y,Z) in the control point in the map .
- Show that the error in the comparing ,is less or more in the Palestinian community and what is the international standard say.

CHAPTER 5

TREVARES ADJSTEMANET

4-1 FIELD SURVEY

Survey has to do with the determination of the relative spatial location of points on or near the surface of the earth. It is the art of measuring slope and horizontal and vertical distances between objects, of measuring angles between lines, of determining the directions of lines, and of establishing point locations by predetermined angular and linear measurements.

With the advance of micro-electronic and computing technology, the use of a total station accompanied by a data logger to capture field data becomes very popular for most land surveyors. Raw data captured from a total station are transmitted to a data logger instantaneously but processing of the data will mainly be done in the office. This procedure can eliminate transcription errors and increase productivity. However, positional and topological errors in captured topographic features cannot be detected until features are formed manually in the office with the aid of a sketch from the field crew. In addition, mistakes can also be made because of incorrect interpretation of the diagram. Very often, re-visit of the site is necessary to solve the problems or re-capture the missing data. Survey data captured will be assigned to an object and associated attribute information can be assigned. Information gathered from this system can easily be loaded into a GIS for revision or updating as we made in our project.

5.1 TRAVERSE

Traverse surveying is one of the most commonly used methods for determining the relative positions of a number of survey points. While leveling is used to establish the elevations of points, traverse surveying is used to determine the horizontal coordinates of these points. Basically, it consists of repeated application of the method of locating by angle and distance. By starting from a point of known position and a line of known direction, the location of a new point is determined by measuring the distances and angle from the known point. Then, the location of another new point is determined by angle and distance measurement from the newly located point. This procedure is repeated from point to point. The resulting geometric figure is called a traverse.

5.2 Purpose of the traverse:

The traverse serves several purposes among, which are:

1. Property survey to establish boundaries.
2. Location and construction layout surveys for highways, railways and other works.
3. Ground control surveys for photogrammetric mapping or map assessment.
4. Map revision of new features like buildings, streets...etc.

5.3 Choice of traverse stations:

Traverse stations should be located so that:

1. Traverse lines should be as close as possible to the boundaries of the tract of land to be surveyed.
2. Distances between stations should be approximately equal and the shortest line should be greater than one third of the longest line.
3. Stations should be located on firm ground.
4. When standing on one station and using Total Station Instrument, it must be easy to see the back sight and foresight stations.

5. Horizontal angle between traverse lines should be neither sharp nor wide-open

Types of traverse:

There are two main types of traverse:

I. Open traverse:

This type originates at a point of known position and terminates at a point of unknown position (figure 5-6). To minimize errors, distances can be measured twice, angles repeated, and magnetic azimuth observed on all lines. This type is used in certain works such as locating the center line of a tunnel during construction. For projects requiring high accuracy, it must not be used.

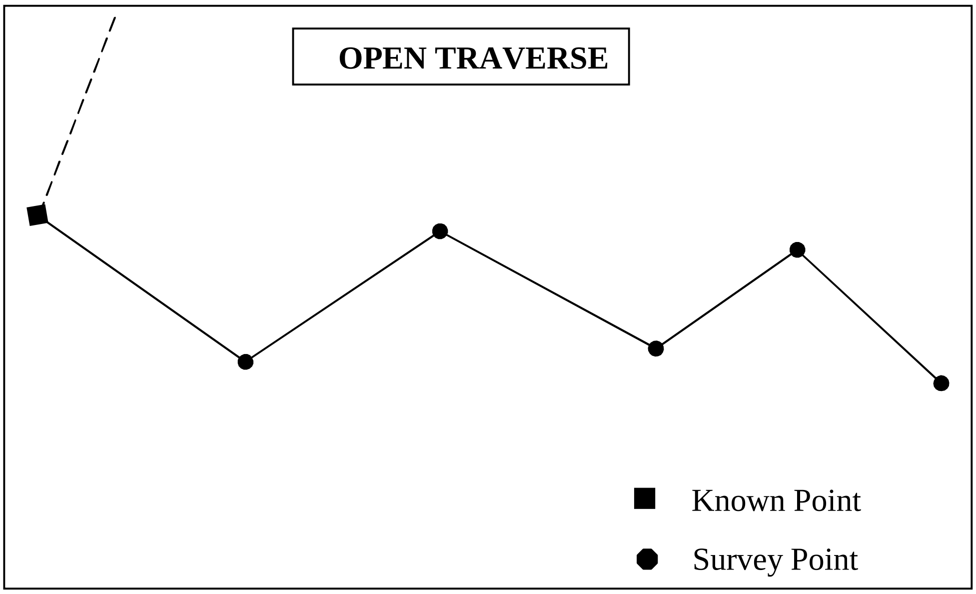


Figure (5-6): Open traverse

II. Close traverse:

This type originates at a point of known position and terminates at the same point or at another point of known position as we made in our project (link traverse) (figure 5-7). This type is preferred to the first type because it provides a check on errors.

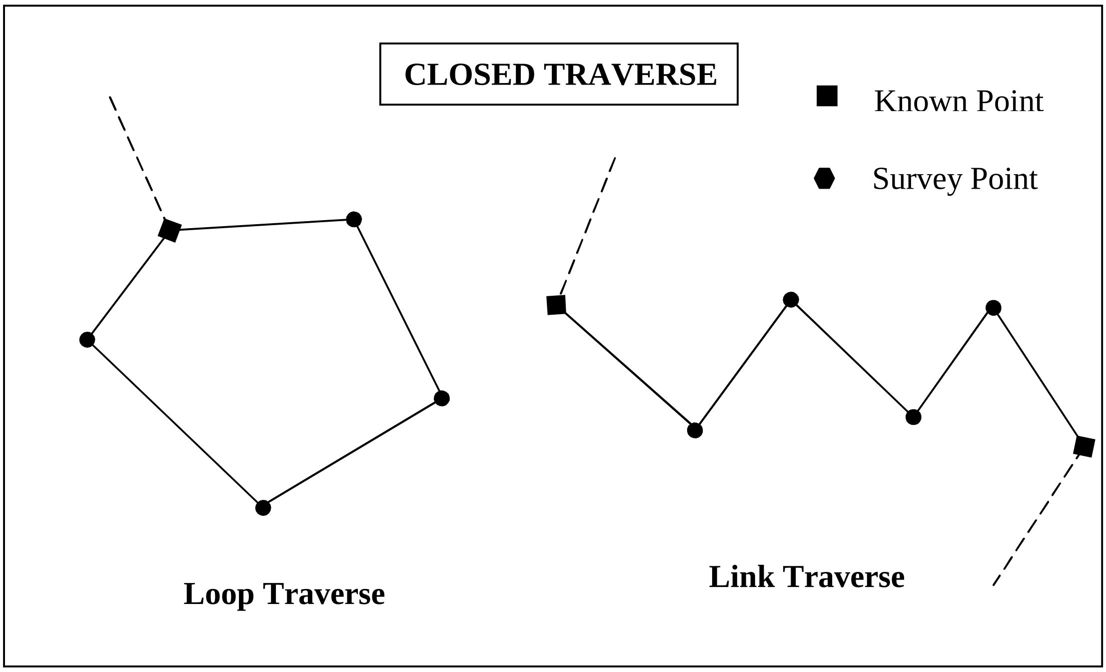


Figure (5-7): Closed traverse

The concept of adjustment

In surveying, the measurements or observations rarely are used directly as the required information. In general, they are used in sub sequent operations to derive, often computationally, other quantities, such as directions, lengths, relative positions, areas, shapes, and volumes. The relationships applied in computational effort are the mathematical representations of the geometric and/or physical conditions of the problem. Such representation, in the context of the present activities, is referred to as the mathematical model. The moment you have observations that must be reduced in some fashion to yield the required useful information’s ,you a model in such a reduction process. Suppose we are interested ion the elevation of tow points, B and C, given and elevation of a third point A. we usually do not measure the elevations directly. Instead, we measure differences in elevation such as l1 from A to B, l2 from B to C, and l3 from C to A, as shown in figure (). The model to this case is relatively simple; consists of the determination of the observations X_1, X_2 of points B, and C relative to a reference level surface (the vertical datum). It is clear that, to determine the model uniquely we need to measure only tow differences in elevation. How ever, because we expect the measurement to contain random errors, we measure more than the necessary minimum, in this caser three, l_1, l_2, l_3. the given elevation E, the three measurements (and their quality as my be expressed by the standard deviation). And the two unknown elevation X_1, X_2, constitute the mathematical model of this problem. To compute X_1, X_2, we need to describe the relationships between the elements of the mathematical model in the form of equations:

$$E + L_1 = X_1$$

$$X_1 + L_2 = X_2$$

$$X_2 + L_3 = E$$

We need to calculate only tow quantities,X_1, X_2, so not all three equations are required to describe the functional model of the problem; two will sufficient. Ion this situation. We have three choices; () and (), or () and (), or() and(). Because of random errors, such of these choices is likely to yield different values for X_1, X_2. the selection of appropriate choice cannot be done arbitrarily, because the surveyor needs to be confidence of the results. There for, when redundant measurements are exist, a means must be found to arrive at asset of values for unknown parameters that are unique consistent with the mathematical model. The procedure used to obtain unique values of the unknown quantities no matter witch sufficient subset of the observations is used to compute them is called data adjustment. The term adjustment is used to imply that the given values of the observations must be altered, or adjusted, to make them consistent with the model. Leading to the uniqueness of the estimated unknowns.

Several techniques can be used to adjust redundant measurements. The most rigorous and commonly used is the method of least squares. Other approximate adjustment techniques have be used by the surveyor. As an example, consider the closure of the level loop in figure (). If all three measurement are consistent, their algebraic some should be 0. usually, this would not be the case and a closure error would result ,or

$l_1 + l_2 + l_3 = c$

Where c is the closure error. If the three observed differences in elevation are of equal quality, or weight, ten, each observation would have a residual equal to (-c/3). Thus,

$(l_1 + v_1) + (l_2 + v_2) + (l_3 + v_3) = (l_1 – c/3) + (l_2 – c/3) + (l_3 – c/3) = l_1 + l_2 + l_3 = 0$

in which v_1, v_2, v_3 are the residuals and l_1, l_2, l_3 are the three adjusted observations. Now because the l_i , are consistent with the model, any two would lead to the same values for X_1 and X_2.

Simple adjustment methods

In the preceding concept of adjustment we gave relatively simple example of a single leveling loop. The error of closure was divided equally among the three measured differences in elevation so that the sum of the adjusted (or corrected) observations became 0, thus eliminating the closure error. The assumption was made in that example that all three differences in elevation were measured along courses of approximately the same length, using the same instruments and observer, and so forth. In short, it was assumed that the three measurements are of equal quality , or weight . if this were not true and the difference weights of the observations could be estimated, then each observation would be assigned residual that is some what different from others.

To illustrate the use of weights, assume that the three differences in elevations, I_1,I_2, I_3, have the weights w1 = 1, w2 = 2, w3 = 2, respectively. These can be loosely interpreted as that we are twice as confident in the quality of I2 and I3 than we are in I2. it follows that I2 and I3 would receive corrections each of which is half that for I1. thus, the proportion of the closure error c assigned to each residual is in the ratio v1: v2: v3 = 1/W1:1/w2: 1/W3 = 1:0.5:05. if the closure error is . say, 12 cm, then

The adjusted observation therefore would be $\hat{1}_1 = I_1$ - 6 cm, $\hat{I}_2 = I_2$ - 3 cm, and $\hat{I}_3 = 1_3$ - 3 cm.

The procedure of distributing closure error, either by simple proportion or by weighted proportion, is used, at least as an approximate adjustment, in different surveying situations. For example, if the three interior angles of a plane triangle are measured and do not add to 180 , the difference is apportioned to the three angles either equally or inversely proportional to their weights, if such weights are available, traverse closure errors also are distributed according to simple rules, as will be described . a single line or loop of levels is adjusted in proportion to the number of instrument setups required or the distance leveled.

COVARIANCE, COFACTOR, AND WEIGHT MATRICES

The one-dimensional case contains one random variable, x, with mean or expectation $«$ *and a variance* $\pm_x^2$. *The two-dimensional case has two random variables, x and y, with means* $«_x$ *and* $«_y$ *and variances* $\pm_x^2$ *and* $\pm_y^2$, *respectively, and covariance* $\pm_{xy}$. *These three parameters can be collected in a square symmetric matrix,* Σ, *of order 2, and called the variance covariance matrix or simply the covariance matrix, It is constructed as*

$$\Sigma = \begin{bmatrix} \pm_x^2 & \pm_{xy}^2 \\ \pm_{xy}^2 & \pm_y^2 \end{bmatrix}$$

(2.29)

where the variances are along the main diagonal and the covariance is off the diagonal. The concept of the covariance matrix can be extended to the multidimensional case by considering random variables $x_1, x_2, \ldots, x_n$ and writing

$$\Sigma = \begin{bmatrix} \pm_1^2 & \pm_{12} & \cdots & \pm_{1n} \\ \pm_{12} & \pm_2^2 & \cdots & \pm_{2n} \\ \cdots & & & \\ \pm_{1n} & \pm_{2n} & \cdots & \pm_n^2 \end{bmatrix}$$

(2.30)

which is an n × n square symmetric matrix.

Often in practice, the variances and covariances are not known in absolute terms hut only W a scale factor. The scale factor, given the symbol $\pm_0^2$, is termed the reference Variance, although other names, such as variance factor and variance associated with weight writs, also have been used. The square root $\pm_0$ of $\pm_0^2$ is called the reference standard deviation, classically known as the standard error of unit weight. The relative variances and covariances, called cofactors, are given by

$$q_{ij} = \frac{\pm_i^2}{\pm_0^2} \quad \text{and} \quad q_{ij} = \frac{\pm_{ij}}{\pm_0^2}$$

(231)

Collecting the cofactors in a square symmetric matrix produces the *cofactor matrix*, **Q,** with the obvious relationship with covariance matrix

$$\mathbf{Q} = \frac{1}{\pm_0^2}\Sigma$$

(2.32)

When **Q** is nonsingular, its inverse is called the *weight matrix* and designated **W**; thus.

$$\mathbf{W} = Q^{-1} = \frac{1}{\pm_0^2}\Sigma^{-1} \qquad (2.33)$$

If $\pm_0^2$ is equal to 1 or, in other words, if the covariance matrix is known, the weight matrix becomes its inverse.

Equation (2.33) should be carefully understood, particularly in view of the classical definition of weights as being inversely proportional to variances. This clearly is not true unless all covarinces are equal to 0, which means that all the random variables are mutually uncorrelated. Only then would Σ (and **Q)** become a diagonal matrix and the weight of the random variable become equal to $\pm_0^2$ divided by its variance.

2.19

VARIANCE -COVARIANCE PROPAGATION

In Section 2.15, we introduced the concept of propagation of errors, where consideration was given to the case of uncorrelated random errors. In this section, we generalize it to both correlated quantities and multiple functions of such quantities. This general case of propagation can be expressed as follows.

Let y be a set (vector) of m quantities, each of which is a function of another set (vector) .v of a random variables. Given the covariance matrix Σ **xx** (or the cofactor matrix **Qxx)** for the variables x, the covariance matrix Σ**yy** or cofactor matrix **Qyy)** for the new quantities y may be evaluated from

$$\Sigma\ yy = Jyx\ \Sigma xx\ \ Jyx^{T}$$

$$Qyy = Jyx\ Qxx\ Jyx^{T}$$

$$Jxx = \begin{bmatrix} \frac{\partial y_1}{\partial x_1} & \frac{\partial y_1}{\partial x2} & \cdots & \frac{\partial y_1}{\partial x_n} \\ \frac{\partial y_2}{\partial x1} & \frac{\partial y_2}{\partial x_2} & \cdots & \frac{\partial y_2}{\partial x_n} \\ \cdots & & & \\ \frac{\partial y_m}{\partial x_1} & \frac{\partial y_m}{\partial x_2} & \cdots & \frac{\partial y_m}{\partial x_n} \end{bmatrix}$$

Equation (2.34) or *(2.35)* is quite general inasmuch as multiple functions in terms of several variables are considered and, more important, no restrictions are imposed on the structure of the given covariance matrix Σxx Therefore, the given random variables in general could be of unequal precision and correlated, so that Σxx would no longer be a diagonal matrix. From the general propagation relationships of Equation (2.34), several relationships could he obtained. First, consider the case of a single function y of several (n) variables $x_1, x_2, \ldots, x_n$ that are uncorrelated and with variance $\pm_1^2, \pm_2^{21}, \ldots, \pm_n^2$, respectively. Equation () would become

$$\pm_y^2 = \begin{bmatrix} \frac{\partial y}{\partial x_1} & \frac{\partial y}{\partial x_2} & \cdots & \frac{\partial y}{\partial x_n} \end{bmatrix} \begin{bmatrix} \pm_1^2 & & & 0 \\ & \pm_2^2 & . & \\ & & . & \\ 0 & & & \pm_n^2 \end{bmatrix} \begin{bmatrix} \partial y/\partial x_1 \\ \partial y/\partial x_2 \\ \cdots \\ \partial y/\partial x_n \end{bmatrix}$$

$$\pm_y^2 = (\partial y/\partial x_1)^2 \pm_1^2 + (\partial y/\partial x_2)^2 \pm_2^2 + \ldots + (\partial y/\partial x_n)^2 \pm_n^2$$

Of course, if the variables *x,* were correlated, Σxx in Equation (2.37) would not be a diagonal matrix and Equation (2.38) would include cross-product terms in all combinations. In such a case, it would be unwise to write the expanded form in Equation (2.38) but instead work directly with the matrix form.

Note that Equations (2.34) through (2.38) are given in terms of variances and covariances of the distributions. However, because such parameters are rarely known in practice, the equations apply equally using sample variances and covariances.

2.20

MATHEMATICAL MODEL FOR ADJUSTMENT

In Section 2. 13 we introduced the notion that measurements are obtained for some, but not all, of the elements *amathmatical model being* the geometric or physical situation at hand. It is important that the reader appreciate the importance of the concept of the mathematical model as well as its adequacy for the surveying problem. Example, suppose we are interested in the area of a rectangular tract of land (assumed reasonably small to use computations in a plane). To compute that area, we need the length *a* and the width *b.* The area will naturally be computed from $A = ab$, which is the "functional" model. Now, it appears that all we have to do is go out with a tape, measure the length and width, each once, and apply the two values into the formula to compute the area. This sounds simple enough until we begin taking a closer look at the problem and analyzing the factors involved in its solution.

In addition to the random and systematic errors in tape measurements as discussed in Section 2.2 (see also Chapter 4), let us also examine the adequacy of the model $A = oh$. This model is formulated on the basic assumption that the parcel of land is exactly rectangular. What if one, or more, of the four angles is checked and differences from the right-angle assumption are discovered? The simple model given no longer could be used; instead, another that more properly reflects the *geometric* conditions should be utilized.

As another example, suppose that we are interested in the *s/mope* of a plane triangle. All that is required for this operation is to measure two of its angles, and the shape of the triangle will be uniquely determined. However, if we were to decide, for safety's sake, to measure all three angles, any attempt to construct such a triangle will immediately show inconsistencies among the three observed angles. In this case the model simply is that the sum of the three angles must equal 18O. If three observations are used in this model, **it** is highly unlikely that the sum will equal exactly l80. Therefore, when redundant observations, or more observations than are absolutely necessary. are acquired, these observations will rarely fit the *model* exactly. Intuitively, and relying on our previous discussion, this results from something characteristic to the observations and makes them inconsistent in the case of redundancy. Of course, we first need to be sure of the adequacy of the model (it is a plane triangle and not spherical or spheroidal, for example). Then, we need to express the quality of the measurements before we seek to adjust the observations to fit the model. These concepts are elaborated on in the following sections.

2.21
FUNCTIONAL AND STOCHASTIC MODELS

We indicated previously that survey measurements are planned with a mathematical model in mind that describes the physical situation or set of events for which the survey is designed. This mathematical model is composed of two parts: a functional model amid a stochastic model. The functional *model* is the more obvious part, because it usually describes the geometric or physical characteristics of the survey problem. Thus, the functional model for the example concerning the triangle discussed at the end of the preceding section *involves the determination of the shape* of a plane triangle through the measurement of interior angles. If three angles in a triangle are available, redundant measurements are present with respect to the functional model. This does not say anything about the properties of the measured angles. For example, in one case, each angle may be measured by the same observer, using the same instrument, applying the same instrument technique, and performing the measurements under very similar environmental conditions. In such a case, the three measured angles are said to be equally “reliable.” But certainly in other cases the resulting measurements will not be of equal quality. In fact, as most practicing surveyors know from experience, measurements always are subject to unaccountable influences that result in variability when observations are repeated. Such statistical variations in the observations are important and must be taken into consideration when using the survey measurements to derive the required information. The stochastic *model* is the part of the mathematical model that describes the statistical properties of all the elements involved in the functional model. For example, in the case of the plane triangle, to say that each interior angle was measured and to give its value is insufficient. Additional information should also be included as to how well each angle was measured and if there is reason to believe in a statistical “correlation” (or interaction) among the angles, and if so, how much. The outcome of having a unique shape for the triangle from the redundant measurements depends both on knowing that the sum of its internal angles is l80~ (the functional model) and knowing the statistical properties of the three observed angles (the stochastic model).

2.22

LEAST-SQUARES ADJUSTMENT

The concept of adjustment was introduced in Section 2.13 and some simple adjustment methods in Section 2.14. A more systematic procedure of adjustment is least squares. which is most commonly used in surveying and geodesy. Most people refer to least squares as an adjustment technique equivalent to estimation in statistics. Although adjustment is not the most precise term, it is appropriate because adjustment is needed when there are redundant observations (i.e., more observations than are necessary to specify the model). In this case the observations given are not consistent with the model and are replaced by another set of estimates, classically called adjusted observations (which also is not a precise term) that satisfy the model. As an example, consider the determination of a distance between two points. The distance may be considered a random variable and, if measured once, would have one estimate, so that no adjustment is needed. On the other hand, if the distance were measured three times, there would likely be three slightly different values, x_1, x_2, and x_3. Since the model concerns a single distance that would be uniquely specified by one measurement, it is obvious that there are two redundant measurements. It also is clear that the situation requires adjustment to have a unique solution. Otherwise, there are several different possibilities for the required distance. We can take any one of x_1, x_2, x_3 or a combination of x_1 and x_2, or x_2 and x_3, or x_1 and x_3, or .x_1, x,, and .x_3. Therefore, having redundant observations makes it possible to have numerous ways of computing the desired values. The multiplicity of possibilities and arbitrariness of choice in obtaining the required information obviously is undesirable. Instead, a process or technique must be found so that one always would get one unique answer, which is derived front the data and is the “best~’ that can be obtained. This is why the relative confidence in, or merit of, the different observations should be taken into account when computing the best estimate, which is defined as the estimate that deviates least from all the observations while considering their relative reliability. This is basically the role of least-squares adjustment.

As another example, consider the case of a plane triangle in which the three angles must add up to l80~. That the three angles must add to 1800 represents a functional relationship that reflects the geometrical system involved in the problem. If the shape and not the size of the triangle is of interest, it is unnecessary to observe the magnitudes of three angles, because two angles will be sufficient to determine the third from the functional relationship just mentioned. However, in practice, the three angles~, , *and*,¥ are measured whenever possible, and their sum likely will be different front 1 80g. Suppose that the sum of the angles exceeds 1800 by 3” of arc. Any two of the three measured angles would give the shape of the triangle, but all three possibilities, in general. Will be different. Therefore, to satisfy the condition that the sum of the angles must be $\mathbf{180^{0}}$**,** the values of the observed angles must be altered. Here, there are numerous possibilities: 3” maybe subtracted from any one of the three angles, perhaps 2” could be subtracted from the largest angle. 1” from the second largest, and nothing from the third angle; or it may appear to be more satisfactory if 1” were subtracted from each angle, assuming that they are equally reliable, taken by the same instrument and observer under quite similar conditions. An alternative criterion to those just given may be to apply alterations that are proportionate to the relative magnitudes of the angles, or the magnitudes of their complements or supplements, or even inversely proportionate to such magnitudes. It is clear, then, that, although adjustment is necessary. The large number of possibilities given are quite arbitrary and a criteria is required in addition to the satisfaction of the functional model of summing the angle to 1800. Such is the least-squares criterion, which is introduced in the next section.

2.23

THE LEAST-SQUARES CRITERION

Let I designate the vector of given observations and ³ the vector of residuals (or alterations). Which when added to I yields a set of new estimates, **I.** that is consistent with the model:

$$\mathbf{I} = \mathbf{I} + ³ \qquad (2.39)$$

The statistical or stochastic properties of the observations are expressed by either the covariance or cofactor matrix Σ or **Q**. respectively, or by the weight matrix W. (Note that $\mathbf{W} = \mathbf{Q}^{-1}$ and $\mathbf{W} = \Sigma^{-1}$ if the reference variance $\pm_0^2$ is equal to unity.) With these variables, the general form of the least-squares criterion is given by

$$¤ = ³^{T} W ³ \rightarrow \text{minimum} \qquad (2.40)$$

Note that ¤ is a scalar, for which a minimum is obtained by equating to 0 its partial derivative with respect to ³ **.** In Equation (2.40). The weight matrix of the observations **W** is not necessarily a diagonal matrix, implying that the observations may be correlated. If the observations are uncorrelated, **W** will be a diagonal matrix and the criterion will simplify to

$$¤ = \sum_{i=1}^{n} w_i ³_i^2 = w_1 ³_1^2 + w_2 ³_2^2 + \ldots + w_n ³_n^2 \rightarrow \text{minimum} \ (2.41)$$

which says that the sum of the weighted squares of the residuals is a minimum. Another, simpler case involves observations that are uncorrelated and of equal weight (precision). for which **W** = **I** and ¤ becomes

$$¤ = \sum_{I=1}^{N} ³_I^2 = ³_1^2 + ³_2^2 + \ldots + ³_n^2 \rightarrow \text{minimum} \ (2.42)$$

The case covered by Equation (2.42) is the oldest and may have accounted for the name *least squares*, because it seeks the "least" sum of the squares of the residuals.
If we refer back to the example of measuring a distance three times and assume that x_1, x_2 and x_3 are of equal precision (weight) and uncorrelated, it can be shown that the $\sum_1^3 {}^2$ is a minimum if the best estimate $\hat{x}$ is taken as the arithmetic mean of the three observations. Similarly. if the three interior angles ~, , *and*,¥ in the plane triangle example have a unit weight matrix, the method of least squares will yield all three residuals equal to — $1''$ Therefore. When each angle is reduced by $1''$. Their sum will be 180^0 and the functional model will be satisfied. These two examples, as well as several others, are worked out in detail in the following section.

2.24

REDUNDANCY AND THE MODEL

Before planning the acquisition phase of surveying data, a general model usually is specified either explicitly or implicitly. Such a model is determined by a certain number of variables and a possible set of relationships among them. Whether or not an adjustment of the survey data is necessary depends on the amount of observational data acquired. A minimum number of independent variables always are needed to determine the selected model uniquely. Such a minimum number is designated n_0. If it measurements are acquired ($n > n_0$) with respect to the specified model, then the redundancy, or (statistical) degrees of freedom, is specified as the amount by which n exceeds n_0. Denoting the redundancy by ***r***,

$\boldsymbol{r = n - n_0}$

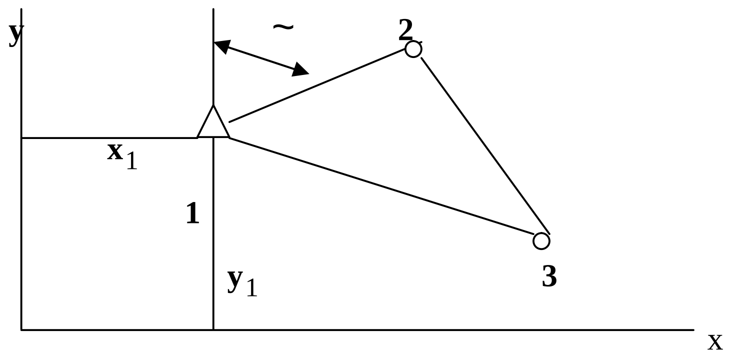
As illustrations, consider the following examples:

1.The shape of a plane triangle is uniquely determined by a minimum of two interior angles, or $n_0 = 2$. If three interior angles are measured, then with n = 3. the redundancy is ***r*** = 1.

2.The size and shape of a plane triangle require a minimum of three observations, at least one of which is the length of one side; or $n_0 = 3$. If three interior angles and all three lengths are available, then with n = 6, the redundancy is ***r*** = 3.

3. In addition to the size and shape of the plane triangle, its location and orientation with respect to a specified Cartesian coordinate system x y also are of interest (Figure 2.6). In this case, the minimum number of variables necessary to determine the model is $n_0 = 6$. which can be explained in one of two ways. From example 2. The size and shape requires that $n_0 = 3$. Then the location of one point (e.g. x_1, and y_1 in Figure 2.6) and the orientation of one side (e.g., ~ in the figure) add three more to make a minimum total of six.

FIGURE 2.6



Another way to determine n_0 is to express the model as simply locating three points, (1, 2. 3 in the figure) in the two-dimensional coordinate system ***x y,*** which obviously requires *six* coordinates. If observations x_1, x_2, ~ are known in addition to the three interior angles and three sides, then with n = 9, the redundancy is ***r*** = 3.

The success of a survey adjustment depends to a large measure on the proper definition of the model and the correct determination of n_0. Next, the acquired measurements must relate to the specified model and have a set that is sufficient to determine the model. If this is not the case, the adjustment would not be meaningful. This can be illustrated by having three different measurements of *one* interior angle in a plane triangle. In this case, even though n = 3 and n_0 = 2, it is clear that the shape of the triangle cannot be determined front these data.

2.25 CONDITION EQUATIONS—PARAMETERS

After the redundancy **r** is determined, the adjustment proceeds by writing equations that relate the model variables to reflect the existing redundancy. Such equations will be referred to either as conditions equations or simply as conditions. The number of conditions to be formulated for a given problem will depend on whether only observational variables are involved or other unknown variables as well. To illustrate this point, consider having two measurements $\sim_1$ and $\sim_2$ for the angle $\sim$. If no additional unknown variables are introduced, there will be only one condition equitant corresponding to the one redundancy. Or $\hat{\sim}_1$ - $\hat{\sim}_2$ = 0. Once the adjustment is performed, the least squares estimate of the angle $\hat{\sim}$ is obtained from another relationship; namely, $\hat{\sim} = \hat{\sim}_1 = \sim_1 + {}^3{}_1 \left(or \hat{\sim} = \hat{\sim}_2 \right)$ Note that this relationship is almost self-evident. Nevertheless, such additional relations are required to evaluate other variables, as will be shown in the following example. As an alternative, a could be carried in the adjustment as an additional unknown variable. In such a case, one more condition must be written in addition to the one corresponding to ***r*** = 1 (i.e., there must be two conditions). These may be written as

$$\hat{\sim} - \hat{\sim}_1 = 0$$

$$\hat{\sim} - \hat{\sim}_2 = 0$$

The additional unknown variable, which is a random variable like the observations, will be called a parameter. The one thing that distinguishes a parameter from an observation is that the parameter has no a priori sample value but the observation does. After the adjustment, both the observations and the parameters will have new least-squares estimates, as well as estimates for their cofactor or covariance matrices, as will be explained in later sections of this chapter.
To summarize, if the redundancy is **r,** there exist **r** independent condition equations, which can be written in terms of the given ii observations. If it additional unknown parameters are included in the adjustment, a total of

$$c = r + u$$

independent condition equations in terms of both the a observations and it parameters must be written. For the parameters to be functionally independent, their number, it, should not exceed the minimum number of variables, n_0. necessary to specify the model. Hence, the following relation must be satisfied:

$$0 \leq u \leq n_0$$

Similarly, for the formulated condition equations to he independent, their number *c* should he no larger than the total number of observations, n. Hence.

$$r \leq c \leq n$$

TECHNIQUES OF LEAST SQUARES

Although there many techniques for least squares a adjustment, we will consider the three most commonly used. It is important to point out, however, that any given survey adjustment problem, all least squares techniques yield identical result. Therefore, the choice of technical depends mainly on the type of problem, the require information, and given to same extent the computing equipment available. Further discussion of this point will be given after the actual technique are presented.

The first technique, called adjustment of indirect observation, is characterized by the following properties:

1. The condition equation include both observation and parameters.
2. There are as many conditions as the number of observation, or c = n.
3. Each condition equation contains only one observation, with the specific stipulation that it's coefficient is unity.

With these properties the condition equation take the functional form

$$v_1 + b_{11}\text{¢}_1 + b_{21}\text{¢}_2 + \ldots + b_{1u}\text{¢}_u = f_1$$
$$v_2 + b_{21}\text{¢}_1 + b_{22}\text{¢}_2 + \ldots + b_{2u}\text{¢}_u = f_2$$
$$\ldots$$
$$v_n + b_{n1}\text{¢}_1 + b_{n2}\text{¢}_2 + \ldots + b_{nu}\text{¢}_u = f_n$$

(4.1)

where $v_1, v_2, \ldots, v_n$ = residuals for the n observation

$\text{¢}_1, \text{¢}_2, \ldots, \text{¢}_u$ = u unknown parameters

$b_{11}, b_{12}, \ldots, b_{un}$ = numerical coefficients of the parameters

$f_1, f_2, \ldots, f_n$ = constant terms for the n conditions, which usually will contain a priori numerical values of the observations.

The test of equation in (1.4) can be collected into matrix form as

$$\begin{bmatrix} v_1 \\ v_2 \\ . \\ . \\ . \\ v_n \end{bmatrix} + \begin{bmatrix} b_{11} & b_{12} & . & . & . & b_{1u} & \text{¢}_1 \\ b_{21} & b_{22} & . & . & . & b_{2u} & \text{¢}_2 \\ & & & & & & . \\ . & . & . & & & & . \\ & & & & & & . \\ b_{1u} & b_{2u} & . & . & . & b_{nu} & \text{¢}_u \end{bmatrix} = \begin{bmatrix} f_1 \\ f_2 \\ . \\ . \\ . \\ f_n \end{bmatrix}$$

(4.2)

or, more concisely,

$$v_{n,1} + B_{n,u}\Delta_{u,1} = f_{n,1}$$

(4.3)

In linear problems, the content-term vector f in equation (4.3) usually is the vector of given observation l subtracted from a vector of numerical constants d.

$$f = d - l$$

(4.4)

and equation (4.3) takes the form

$$v + B\Delta = d - l$$

(4.5)

The second least square technique to be considered is called *adjustment of observation only*. As its name implies, no parameters are in the condition equation, which therefore must be equal in number to the redundancy r. They take the general functional form.

$$a_{11}v_1 + a_{12}v_2 + ... + a_{1n}v_n = f_1$$
$$a_{21}v_1 + a_{22}v_2 + ... + a_{2n}v_n = f_{21}$$
$$...$$
$$a_{r1}v_1 + a_{r2}v_2 + ... + a_{rn}v_n = f_n$$

(4.6)

which in the matrix notation become

$$\begin{bmatrix} a_{11} & a_{12} & . & . & . & a_{1n} \\ a_{21} & a_{22} & . & . & . & a_{2n} \\ . & & & & & \\ . & & & & & \\ . & & & & & \\ a_{r1} & a_{r2} & . & . & . & a_{rn} \end{bmatrix} \times \begin{bmatrix} v_1 \\ v_2 \\ . \\ . \\ . \\ v_n \end{bmatrix} = \begin{bmatrix} f_1 \\ f_2 \\ . \\ . \\ . \\ f_n \end{bmatrix}$$

(4.7)

or, more concisely,

$$A_{r,n}v_{n,1} = f_{r,1}$$

(4.8)

For linear adjustment problems, the constant-term vector f in equation (4.8) is given by.

$$f = d - Al$$

(4.9)

in which d is a vector of numerical constant. Then, equation (4.8) becomes

$$Av = d - Al$$

(4.10)

Adjustment of indirect observations (Parametric adjustment).

To appreciate the matrix derivation to follow, consider the level network. Also, to keep the introductory treatment uncomplicated, assume that the observed differences in elevation are uncorrelated and have the weight $w_1, w_2, ..., w_n$, respectively. Under this assumption, the minimum criterion applies:

$$¤ = w_1v_1^2 + w_2v_2^2 + ... + w_nv_n^2$$

(4.11)

which, in view of the equation

$$W = \begin{bmatrix} w_1 & & & & & \\ & w_2 & & & & \\ & & \cdot & & & \\ & & & \cdot & & \\ & & & & \cdot & \\ & & & & & w_n \end{bmatrix}$$

(5.12)

Recalling matrix B form equation (5.3).

$$(B^TWB)\Delta = B^TWf$$

(5.13)

the relation in equation (5.13), although derived for an example, is an fact general and applies to any problem for which condition equations are of the general form given in equation (5.3). Furthermore, there is no restriction on the structure of the weight matrix W. The following derivation shows that this is true.

The least-squares criterion is

¤ $= v^T W v$

(5.14)

which, upon substituting for equation (5.3) becomes

¤ $= (f - B\Delta)^T W (f - B\Delta)$

¤ $= f^T W f - \Delta^T B^T W f - f^T W B\Delta + \Delta^T B^T W B\Delta$

(4.15)

Because all terms on the right side of equation (5.15) are scalar, the second and third terms are equal and thus

¤ $= f^T W f - 2 f^T W B\Delta + \Delta^T B^T W B\Delta$

(5.16)

For ¤ to be a minimum, ¢¤/¢Δ must be zero, or

$$- 2 f^T W B + 2\Delta^T B^T W B = 0$$

or

$$(B^T{}_{u,n} W_{n,n} B_{n,u})\Delta_{u,1} = B^T{}_{u.n} W_{n,n} f_{n,1}$$

(5.17)

which is identical to equation (5.13).note that W could be a full matrix and therefore no restriction is placed on the statistical of the observations. Of course, certain assumptions about the structure of W may be made practice. If the auxiliaries

$$N = B^T W B$$
$$t = B^T W f$$

(5.18)

are used, equation (5.17) becomes

$$N\Delta = t \qquad (5.19)$$

the solution of which (assuming a nonsingular N) is

$$\Delta = N^{-1}t$$

(5.20)

The set of equation in (5.17) or (5.19) usually called the *reduced normal equations*, or simply the normal equations in the parameters, N is a nonsingular matrix when B is of full rank u, which is usually the case. The matrices N and t in (5.18) are called the normal equations coefficient matrix and the normal equation constant term vector, respectively.

The precision of the estimated parameter of the cofactor matrix $Q_{\Delta\Delta}$. This matrix obtained by applying the relationships of error propagation developed in equation (5.20). Using equation (5.18) for t and equation (5.4) for f ,equation (5.20) becomes

$$\Delta = N^{-1}B^{T}W(N \_ l)$$

(5.21)

the only vector of random variable on the right side of the equation (5.21) is l , since d is a vector of numerical constant. The matrix Q is used to designate the cofactor of the observation.

and therefore when equation (5.22) is applied to equation (5.21), the following results:

$$Q_{\Delta\Delta} = J_{\Delta l} QJ^{T}{}_{\Delta l} = J_{\Delta l} QJ^{T}{}_{\Delta l}$$

$$Q_{\Delta\Delta} = (\_ N^{-1}B^{T}W)Q(NB^{T}W)^{T}$$

or

$$Q_{\Delta\Delta} = N^{-1}B^{T}WQWBN^{-1}$$

noting that N^{-1} and W are symmetric matrices. Because $W = Q^{-1}$,using equation (5.18) for N ,

$$Q\Delta\Delta = N^{-1}(B^{T}WB)N^{-1} = N^{-1}NN^{-1}$$

or

$$Q_{\Delta\Delta} = N^{-1}$$

(5.23)

once Δ is computed (from equation(5.19)), the observational residuals may be computed using equation (5.3) or

$$v = f \_ B\Delta$$

(5.24)

and the least-square estimate of the observation, $\hat{l}$, is evaluated from equation (5.16):

$$\hat{l} = l + v$$

(5.25)

In a manner similar to that used to obtain $Q_{\Delta\Delta}$, a relationship for the cofactor matrix Q_{ii} may be derived as

$$Q_{ii} = BN^{-1}B^{T}$$

(5.26)

If, originally, the covariance matrix of the observation Σ was given and used in the least-square solution instead of the cofactor matrix Q, then $\Sigma = N^{-1}$ and $\Sigma = BN^{-1}B^{T}$, instead of the equations (5.23) and (5.26). On the other hand, if only relative variances and covariance's were given a prior, equation (5.23) and (5.26) may be used to compute $Q_{\Delta\Delta}$ and Q_{ii} . Then, to get Σ and Σ , an estimate $£_0^2$ of the reference variance may be computed from the adjustment using the relationship.

$$Q_{ii} = \frac{v^{T}Wv}{r}$$

(5.27)

In which r is the redundancy, v is the vector of residuals competed from equation (5.24), and W is the a priori weight matrix of the observations. Then, according to equation (2.32)

$$\Sigma = \pm \sigma_0^2 Q_{\Delta\Delta}$$

(5.28)

$$\Sigma = \pm \sigma_0^2 Q_{ii}$$

(5.29)

This result is rather interesting, because it shows that the least-square estimate of the distance using three direct and uncorrelated observations of equal weights is their arithmetic mean, which is what one would intuitively have taken as the **best estimate** of the distance. Although only three observations are considered, the result in general and may be applied to any number of observations. Thus, for **n** observations that are uncorrelated and of equal precision (weight),

$$N = n$$

(5.30)

$$t = l_1 + l_2 + ... + l_n = \Sigma$$

(5.31)

$$\vec{l} = \frac{1}{n}\Sigma$$

(5.32)

and from equation (5.23),

$$q_{ii} = Q_{\Delta\Delta} = N^{-1} = \frac{1}{n}$$

(5.33)

Therefore, the weight of the arithmetic mean of n uncorrelated observations, each of unit weight, is equal to n, because

$$w_{ii} = \frac{1}{q_{ii}} = n$$

(5.34)

In general may be extended to any number of observation, n:

$$N = w_1 + w_2 + ... + w_n = \sum_{i=1}^{n} w_i$$

(5.35)

$$t = w_1 l_1 + w_2 l_2 + ... + w_n l_n = \Sigma$$

(5.36)

$$\vec{l} = \frac{\Sigma}{\Sigma}$$

(5.37)

$$q_{\vec{l}\vec{l}} = Q_{\Delta\Delta} = N^{-1} = (\Sigma \quad)^{-1}$$

(5.38)

Equation (5.37) implies that the weight of the weighted mean of a set of uncorrelated observation (with different weights) is equal to the sum of their weights . It can be seen then the equation (5.34) is the special case of equation (5.37), in which the weight are all equal to unity.

ADJSTMENT OF OBSERVATION ONLY (CONDITIONAL ADJSTMENT)

Similar to the case of indirect observations, this technique is introduced by working the level-network, assuming that we have five observation are uncorrelated and have the weights w_1, w_2, w_3, w_4, w_5 . With this information the minimum criterion is .

$$¤ = w_1 v_1^2 + w_2 v_2^2 + w_3 v_3^2 + w_4 v_4^2 + w_5 v_5^2 \rightarrow \min nimum$$

As example, The two condition for the level network.

$$v_1 + v_2 - v_3 = -l_1 - l_2 + l_3 = f_1$$
$$v_3 + v_4 - v_5 = -l_3 - l_4 + l5 = f_2$$

(5.39)

Unlike the adjustment of indirect observations, it is not possible here to substitute for the residual form the condition equations because there are five and only two condition equations in equation (5.39). Therefore, in this case, a minimum for function ϕ is sought under the constraint imposed by the condition equations. This makes the problem that seeking a *constrained minimum* instead of a free minimum, as it is termed in mathematics. Such a constrained minimum is obtained most conveniently by adding (algebraically) to ¤ each of the condition equations multiplied by a factor λ. This factors are called *Lagrange multipliers*. It is numerically more convenient for our later development to use *2***k** instead λ; therefore, the function to be minimized becomes

$$\phi = w_1 v_1^2 + w_2 v_2^2 + w_3 v_3^2 + w_4 v_4^2 + w_5 v_5^2 - 2K_1(v_1 + v_2 - v_3 - f_1)$$
$$- 2K_2(v_3 + v_4 - v_5 - f_2)$$

(5.40)

Note that, after the adjustment, the quantities within parentheses in (5.40) vanish because the two condition equation (5.39) are fully satisfied after the adjustment. Consequently, the minimum of $\bar{\phi}$ corresponds to the minimum of the original function ϕ. Taking the partial derivatives of $\bar{\phi}$ with respect to each of the five residuals and equating to 0 leads to

$$\frac{\partial \bar{\phi}}{\partial v_1} = 2w_1 v_1 - 2k_1 = 0$$
$$\frac{\partial \bar{\phi}}{\partial v_1} = 2w_2 v_2 - 2k_1 = 0$$
$$\frac{\partial \bar{\phi}}{\partial v_1} = 2w_3 v_3 + 2k_1 - 2k_1 = 0$$
$$\frac{\partial \bar{\phi}}{\partial v_1} = 2w_4 v_4 - 2k_2 = 0$$
$$\frac{\partial \bar{\phi}}{\partial v_1} = 2w_5 v_5 - 2k_2 = 0$$

(5.41)

partial differentiation of $\bar{\phi}$ with respect to k_1 and k_2 and equating the result to 0 yield the two condition equation in (5.39). Therefore, combining equation (5.41) and (4.39) result in seven linear equations with seven unknowns: $v_1, v_2, v_3, v_4, v_5, k_1, k_2$ Solving equation (5.41) for the five residuals yields

$$v_1 = \frac{1}{w_1} k_1$$
$$v_2 = \frac{1}{w_2} k_1$$
$$v_3 = \frac{1}{w_3} (-k_1 + k_2)$$
$$v_4 = \frac{1}{w_4} k_2$$
$$v_5 = \frac{1}{w_5} - k_2$$

which in the matrix form become

$$\begin{bmatrix} v_1 \\ v_2 \\ v_3 \\ v_4 \\ v_5 \end{bmatrix} = \begin{bmatrix} \frac{1}{w_1} & & & & 0 \\ & \frac{1}{w_2} & & & \\ & & \frac{1}{w_3} & & \\ & & & \frac{1}{w_4} & \\ 0 & & & & \frac{1}{w_5} \end{bmatrix} \begin{bmatrix} 1 & 0 \\ 1 & 0 \\ -1 & 1 \\ 0 & 1 \\ 0 & -1 \end{bmatrix} \begin{bmatrix} k_1 \\ k_2 \end{bmatrix} \qquad (5.42)$$

the first (diagonal) matrix on the right side of equation (5.42) is the inverse of the weight matrix, or W^{-1}, which is equal to the cofactor matrix of the observations, Q. The second matrix is $\mathbf{A^T}$ **,** equation (5.42) may be written more concisely as:

$$v = QA^T k$$

(5.43)

in which all the terms are as defined and **k** is the vector of Lagrange multipliers, or $k = [k_1 \quad k_2]^T$. Substituting for v form equation (5.43) into the condition equations of givens

$$AQA^T k = f$$

(5.44)

which may by solved for k as

$$k = (AQA^T)^{-1} f$$

(5.45)

Finally, substitute the value of k computed from equation (5.45) in to equation (5.43) to get values for the residuals.

The relations (5.44) and (5.45) are not specific for this particular example but are general for this technique of least-square *adjustment of observations* only as shown in the following derivation. Let $k_{r,1}$ be vector of r Lagrange multipliers, one for each of the **r** condition equation in (5.8) . Then the function to be minimized is

$$\bar{\phi} = v^T W v - 2k(Av - f)$$

For $\bar{\phi}$ to be minimum, $\delta\bar{\phi}/\delta\mathbf{v}$ must be 0, or

$$\frac{\partial\bar{\phi}}{\partial v} = 2v^T W - 2k^T A = 0$$

which, after transposing and rearranging, becomes

$$Wv = A^T k$$

And solving for **v** yields

$$\underset{n,1}{v} = W^{-1} A^T k = \underset{n,n}{Q}\ \underset{n,r}{A^T}\ \underset{r,i}{k}$$

(5.46)

substituting equation (5.46) into equation (5.8) yields

$$(AQA^T)k = f$$

(5.46a)

which when using the auxiliary

$$\underset{r,r}{Q_e} = \underset{r,n}{A}\ \underset{n,n}{Q}\ \underset{n,r}{A^T}$$

(5.47)

Leads to

$$Q_e k = f$$

or

$$k = Q_e^{-1} f = W_e f$$

(5.48)

The matrix Q_e can be considered the cofactor matrix foe an equivalent set of observations, $\underset{r,1}{l_e}$, containing as many observations as there are condition equation. Because $r < n$, the number of equivalent observations always is less than the number of original observations. Each equivalent observation is a linear combination of the original observations. The linear relations are expressed by the matrix A; therefore

$$l_e = Al$$

(5.49)

By error propagation, then, the cofactor matrix Q_e for l_e may be evaluated as

$$Qe = J_{l_e l} Q J_{l_e l}^T = AQA^T$$

(5.50)

which is identical to equation (5.47). The inverse of Q_e is designated W_e, as shown in equation (5.48).

The final relation for $\mathbf{v}$ is obtained by substituting for $\mathbf{k}$ from equation (5.48) into equation (5.47):

$$v = QA^T W_e f$$

(5.51)

Precision estimation after the adjustment may be performed using the rustle of error propagation. The estimated observations, $\hat{\mathbf{l}}$, are given by equation (5.25):

$$\hat{l} = l + v = l + QA^T W_e f$$

which, from equation (5.4), becomes

$$\hat{l} = l + QA^T W_e (d - Al)$$

(5.52)

Applying equation (2.34) to equation (5.52) results in

$$Q_{\hat{l}\hat{l}} = J_{\hat{l}\hat{l}} Q Q J_{\hat{l}\hat{l}}^T$$
$$= (l - QA^T W_e A) Q (l - QA^T W_e A)^T$$
$$== (Q - QA^T W_e AQ)(l - A^T W_e AQ)$$

or

$$Q_{\hat{l}\hat{l}} = Q - QA^T W_e AQ - QA^T W_e AQ + QA^T W_e AQA^T W_e AQ$$

From definition of Q_e in equation (5.47) and the fact that $W_e - Q_e^{-1}$, the last term reduces to the negative of the third term, and therefore the two cancel out and the final expression for the cofactor matrix of the estimated observations becomes

$$Q_{\hat{l}\hat{l}} = Q - QA^T W_e AQ$$

(5.53)

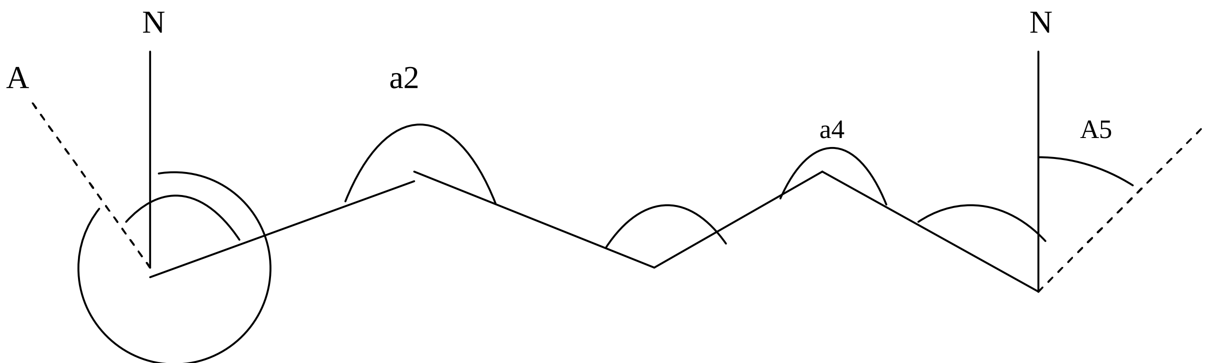
The reader should evaluated the cofactor matrix of the residuals Q_{vv} from equation (5.51) and verify that it is negative of the last term in equation (5.53). So, an alternative to equation (5.53) is

$$Q_{\hat{l}\hat{l}} = Q - Q_{vv}$$

(5.54)

TRAVERSE ADJUSTMENT BY LEAST DQUARE S

A traverse is composed consecutive distance and angle measurement, Figure 3.1 shows a traverse between two horizontal control points, A traverse can be adjusted using either of the two techniques of least square presented in chapter 4. The technique of *least square adjustment of indirect observations* is applied more frequently in practice.



a1 2 a3 4 a5

l_{12} l_{23} 3 l_{34} l_{45}

1 5

Figure 3.1

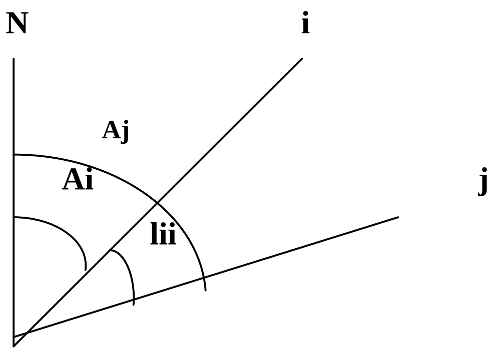


figure 3.2

There are two types of condition equations: the angle condition, and distance condition. To derive the condition equation for the angle l_{ij} in figure 3.2, first write the two azimuth condition equation the for azimuth A_i and A_J :

$$A_i = \arctan \frac{X_i - X_p}{Y_i - Y_p}$$

(5.1)

$$A_j = \arctan \frac{X_j - X_p}{Y_j - Y_p}$$

(5.2)

Then, the angle condition equation for the angle l_{ij} is

$$v_{ij} + l_{ij} = A_j - A_i = \arctan\frac{X_j - X_p}{Y_J - Y_P} - \arctan\frac{X_I - X_P}{Y_I - Y_P}$$

or

$$v_{ij} + l_{ij} + \arctan\frac{X_j - X_p}{Y_J - Y_P} - \arctan\frac{X_I - X_P}{Y_I - Y_P} = 0$$

(5.3)

Equation (5.3) obviously is nonlinear in the parameter and must be linearized. Recall that.

$$\frac{¢}{¢x}\arctan u = \frac{1}{1+u^2}\frac{¢u}{¢x}$$

(5.4)

and write equation (5.3) in a function form:

$$v_{ij} + l_{ij} + F_{ij}(X_p, Y_p, X_i, Y_i, X_j, Y_j) = 0$$

(5.5)

The linearized form of the angler condition equation is

$$v_{ij} + l_{ij} + F_{ij}(X_p, Y_p, X_i, Y_i, X_j, Y_j)^0 + \frac{¢F_{ij}}{¢X_p}¢X_p + \frac{¢F_{ij}}{¢Y_p}¢Y_p + \frac{¢F_{ij}}{¢X_i}¢X_i + \frac{¢F_{ij}}{¢Y_i}¢Y_i + \frac{¢F_{ij}}{¢X_j}¢X_j + \frac{¢F_{ij}}{¢Y_j}¢Y_j = 0$$

or

$$v_{ij} + b_1¢X_p + b_2¢Y_p + b_3¢X_i + b_4¢Y_i + b_5¢X_j + b_6¢Y_j = -l_{ij\_F_{ij}}\left[(X,Y)_p^0, (X,Y)_i^0, (X,Y)_j^0\right] = f_{ij}$$

(5.6)

Equation (5.4) may be applied to the angle condition given in equation (5.3) to evaluate the coefficients $b_1, b_2, \ldots, b_6$ in equation (5.6).As example the partial derivatives with respect to X_p and Y_p are:

$$b_1 = \frac{¢F_{ij}}{¢X_p} = \frac{-(Y_i - y_p^0)}{(Y_i - y_p^0)^2 - (X_i - X_p^0)^2} - \frac{-(Y_j - y_p^0)}{(Y_j - y_p^0)^2 - (X_j - X_p^0)^2} \qquad (5.7a)$$

$$b_2 = \frac{¢F_{ij}}{¢X_p} = \frac{X_i - X_p^0}{(Y_i - y_p^0)^2 - (X_i - X_p^0)^2} - \frac{X_j - X_p^0}{(Y_j - y_p^0)^2 - (X_j - X_p^0)^2} \qquad (5.7b)$$

and the right-hand side is given by

$$f_{ij} = -l_{ij} - \arctan \frac{X_i - X_p^0}{Y_i - Y_p^0} + \arctan \frac{X_j - X_p^0}{Y_j - Y_p^0}$$

(5.8)

f_{ij} = (calculated angle) – (measured angle)

The distance condition expresses the distance between two point i and j as a function of their coordinates, or

$$l + v - \left[(X_i - X_j)^2 + (Y_i - Y_j)^2 \right]^{\frac{1}{2}} = 0$$

(5.9)

Because equation (5.9) is nonlinear, it must be linearized using a Taylor by first writing it in the form

$$l_{ij} + v_{ij} + F_{ij}(X_i, Y_i, X_j, Y_j) = 0$$

(5.10)

then applying

$$v_{ij} + b_1 dX_i + b_2 dY_i + b_3 dX_j + b_4 dY_j = -l_{ij} + l_{ij}^0 = f_{ij}$$

(5.11)

in which

$$b_1 = \frac{\partial F_{ij}}{\partial X_i} = -\frac{X_i^0 - X_j^0}{l_{ij}^0} \qquad (5.12a)$$

$$b_2 = \frac{\partial F_{ij}}{\partial Y_i} = -\frac{Y_i^0 - Y_j^0}{l_{ij}^0} \qquad (5.12b)$$

$$b_3 = \frac{\partial F_{ij}}{\partial X_i} = \frac{X_i^0 - X_j^0}{l_{ij}^0} = -b_1$$

(5.12d)

$$b_4 = \frac{\partial F_{ij}}{\partial Y_i} = \frac{Y_i^0 - Y_j^0}{l_{ij}^0} = -b_2$$

(5.12c)

with

$$l_{ij}^{0} = \left[(X_i^0 - X_j^0)^2 + (Y_i^0 - Y_j^0)^2\right]^{\frac{1}{2}}$$

(5.7e)

CHAPTER THREE

ERRORS

3.1 INTRODUCTION

Measurements are essential to the functions of the surveyor. The surveyors task is to design the survey; plan its field operations; designate the amount, type, and acquisition techniques of the measurements; and then adjust and analyze these measurements to arrive at the required survey results. It is important, then, that the individual studying surveying understand the basic idea of a measurement or an observation (here, the two terms are used interchangeably).

Except for counting, measuring entails a physical operation that usually consists of several more elementary ones, such as preparation (instrument setup, calibration, or both), pointing, matching, and comparing. Yet, the result of these physical operations is assigned a numerical value and called the measurement. Therefore, it is important to note that a measurement really is an indirect thing, even though in some simple instances it may appear to be direct. Consider, for example, the simple task of determining the length of a line using a measuring tape, this operation involves several steps: setting up the tape and stretching it, aligning the zero mark to the left end of the line, and observing the reading on the tape at the right end. The value of the distance as obtained by subtracting 0 from the second reading is what we call the measurement, although actually two alignments have been made, to make this point clear, visualize the tape simply aligned next to the line and the tape read opposite to its end. In this case, the reading on the left end may be 11.9 and the one on the right end 52.2, with the net length measurement of 40.2.

Amore common situation regarding observations involves the determination of an angle. This raises the question of distinguishing between directions and angles. Directions are more fundamental than angles, since an angle can be derived from two directions. Where both directions and angles may be used, care must be exercised in treating each group, because an angle is the difference between two directions. As in measuring a line, an angle is obtained directly if the first pointing (or direction) is taken specifically at 0 then observation. However, if the surveying process is formulated to be fundamentally in terms of directions, the values read for these directions then should be the observations. In this case, any angle determination will be a simple linear function of the observed directions and its properties can be evaluated from the properties of the directions and the known function, as will be sho wn later.

3.2 OBSERVATIONS AND ERRORS

Let us consider the measurement of the length of a tract of land. Begin by assuming that a standardized tap is available (which may not be true!). to make a length measurement, we have to align the zero mark and note that reading at the other end of the line, being human beings working with a manufactured tool (the tape), we have no reason that our determination of length is the best value we can obtain. We at least are uncertain about whether this one measurement in fact will be the best we can do.

Perhaps. If w e repeat the measurement twice, we would feel more confident. And if we do, we probably will end up with two different values for the length. We have no reason to accept one measurement over the other; both appear equally reasonable.

Return to the assumption that the tape is standardized. What if the tape is too short by a certain amount and we do not know it? Would not all our measurements with that tape be too large by factors of the amount by which the tape is short? When the measured length is used directly in computing an area, it obviously will be incorrect.
It is clear, then , that variability in repeated measurements (under similar conditions) is an inherent quality of physical processes and must be accepted as a basic property of observations. Therefore, observations or measurements are numerical values for random variables, which are subject to random fluctuations . Once this is recognized, we may proceed to treat the observations employing established statistical techniques to derive estimates or make appropriate inferences. The term error in general, can be considered to refer to the difference between a given measurement and the true or exact value of the measured quantity. It will always exist, because the repeated measurements will vary naturally, whereas the true value remains a constant.

3.3 TYPES OF ERRORS IN DATA COLLECTION

An error is a difference between the true value and the observed value of a quantity caused by the imperfection of equipment, by environmental effects, or due to the imperfection in the senses of the observer.
Errors are generally classified into three types:

1. Mistakes or blunders.
2. Systematic errors .
3. Random errors.

3.3.1 Mistakes

- are caused by carelessness or inattention of the observer in using equipment, reading scales or dials or in recording the observations. For example ,the observer may bisect the wrong target in angle observations. or may record observations by transposing numbers ,e.g. by writing 65.25 instead of 56.25. They could also be introduced by misidentification of a control point in an aerial photograph .Gross errors may also be caused by failure of equipment. Observation fraught with gross errors are useless .Therefore ,every attempt must be made to eliminate gross errors .Normally, observation procedures are designed in such a way that one can detect gross errors during or immediately after the observations are taken .
- some of the techniques used in ducting and eliminating gross errors include taking multiple reading on scales and checking for consistency using simple geometric and algebraic checks, repeating the whole measurement and checking for consistency ,etc.
- In a statistic sense ,gross errors are observations which cannot be considered to belong to the same sample as the rest of the observations. Therefore, the elimination of gross errors or blunders or mistakes is vitally important .

3.3.2 Systematic errors

- occur in accordance with some deterministic system which, if known, may be represented by some functional relationship. For example, observed slope distances if not reduced to the ellipsoid will introduce systematic errors .
- There is functional relationship between the observed distance, geoids ellipsoid separation ,and the heights of the points between which the distances observed. In surveying, geodesy, and photogrammetry systematic error's occur because of environmental effects, instrumental imperfections, and human limitations.
- Some of the environmental effects are humidity, temperature, and pressure changes. These factors affect distance measurements, angle measurements, and GPS satellite observation, among others. Instrumental effects include lack of proper calibration and adjustment of the instrument as well as imperfection in the construction of the instrument, e.g., nouniform graduations of the linear and

circular scales. Systematic errors must be detected and observations must be corrected for systematic errors or they must be modeled by me mathematical model.

- In a statistical sense, systematic errors introduce bias in the Observations. Unlike gross errors, they cannot be detected or eliminated by repeated observations. Therefore, if systematic errors are present ,the measurements may be precise but they will not be accurate.

3.3.2.1 SYSTEMATIC ERROR CORRECTIONS AND RESIDUALS

Classically, if an error is removed from a measurement, the value of that measurement should improve. This idea is applicable only to systematic errors. Thu if x is an observation and e_x is a systematic error in that observation

$$x_e = x - e_x$$

Is the value of the observation corrected for the systematic error, again , in the classical error theory, a correction c_s for systematic error was taken equal in magnitude but opposite in sign to the systematic error; therefore,

$$x_c = x + c_s$$

In fact, the concept of error and correction is not limited to the systematic type but is regarded as general in nature, with x_c termed the true value, which is never known. More realistically, the observations are sample values from a random probability distribution with mean « and some specified variation like the true value , the distribution mean « usually is unknown, and we can make only an improved estimate for the observation. The improved estimate for an observation x is designated $\overline{X}$, and instead of error or correction, we use the term residual ^{3} , such as

$$\overline{X} = x + {}^{3}$$

The residual , which will be used throughout this development, although having the same sign sense as the correction. Is not a hypothetical concept, because it has value in the reduction observational data.

3.3.2.2 QREPROCESSING (EXAMPLES OF COMPENSATION FOR SYSTEMATIC ERRORS)

Preprocessing of survey measurements involves both the elimination of blunders and correction for all known systematic errors, examples of a variety of systematic errors abound in geodetic and surveying operations. Consider the operation of measuring by tape to determine distances between points on the earths surface. The length ofagiven tape may be physically different from the values indicated by the numbers written on its graduations. Owing to some or all of the following factors:

1. The temperature has changed between that used for tape standardization (calibration) and the temperature actually recorded in the field during the observation the tension or pull applied to the tape during measurement is different from that used during calibration
2. The method of tape support is different during measurement from that used during calibration.
3. The endpoints of the distance to be measured are at different elevations and the horizontal distance is desired . in this case a correction I needed because one would be measuring the slope distance instead of the horizontal distance.

electronic (and electro-optical) distance measuring techniques also are subject to a number of systematic effects whose sources and characteristics must be determined and alleviated. Of these sources the following are mentioned here:

1. The density of air through which the signal travels may change and cause a change and cause a change in the signal frequency (due to variations in wave propagation velocity) .
2. The instrument (and sometimes the reflector or remote unit) may not be properly centered on the ends of the line to be measured. The path of propagation of the signal may not conform to the straight-line assumption and may be deviated due to environmental and other factors.

3.3.3 RANDOM ERRORS

- After all the gross errors and systematic errors are removed there will still remain some variants in the observations .These remaining variations in observations (which are small in magnitude) are called random errors. They cannot be represented by a functional relationship based on a deterministic model. Random errors occur due to the imperfection of the instrument and observed quantity will be either too small or too large every time it is observed. If sufficient observations are taken.

random errors possess the following characteristics :

- Positive and negative errors occur with the same frequency.
- Small errors occur more often than large errors.
- Large errors rarely occur.

Random errors are treated systematically by using a stochastically model.

After mistakes are detected and eliminate and all sources of systematic errors are identified and corrected , the values of the observations will be free of any bias and regarded as sample values for random variables.

A random variable may be defined as a variable that takes on several possible values and a probability is associated with each value.

Probability may be defined as the number of chances for success divided by the total number of chances or as the limit value to which the relative frequency of occurrence tends as the number of repetitions is increased indefinitely. As a sample illustration, consider the experiment of throwing a die and noting the number of dots on the top face. That number is a random variable , because there are six possible values, from 1 to 6. for example, the probability that three dots will occur tends to be 1/6 as the number of throws gets to be very large. This probability value of 1/6 or 0.166 is the limit of the relative frequency. Or the ratio between the number of times three dots show up and the total number of throws.

A survey measurement. Such as a distance or angle, after mistakes are eliminated and systematic errors corrected, is a random variable such as the number of dots in the die example. If the nominal value of an angle is $41^{o}13'36''$ and the angle is measured 20 times, it is not unusual to get values for each of the measurements that differ slightly from the nominal angle. Each of these values has a probability that it will occur. The closer the value approaches $41^{o}13'36''$ the higher is the probability; and the farther away it is ,the lower is the probability.

In the past, the value $41^{o}13'36''$ was designated the " true value," which was never known. Then, when an observation was given that, owing to random variability, was different from the true value, an error was defined as

error = measured value – true value

and a correction, which is the negative of the error, was defined as

correction = true value – measured value

Whereas for systematic effects, the concept of error and correction is reasonable, in the case of random variation it is not. Since there is no reason to say that errors have been committed . the so–called random error actually is nothing but a random variable, because it represents the difference between a random variable, the measurement, and a constant, the true value. The ideal value of the error is 0 (which in statistics is called the expected value): that for the observation is the true value (in statistics called the expectation or distribution mean « , as will be explained later. The variation of the random errors around – is identical to the variation of the observations around the expectation « – or the true value . thus, it is better to talk about the observations themselves and seek better estimates for these observations than discuss errors, because, strictly speaking, the values being analyzed are not error. Than discuss errors, because, strictly speaking, the values being analyzed are not errors.

Another classically used term is discrepancy, which is the difference between two measurements of the same variable. Best value, most probable value, and corrected value are all terms that refer to a new estimate of a random variable in the presence of redundancy. Or having more measurements than the minimum necessary. Such and estimate usually is obtained by some adjustment technique, such as least squares, if the random variable is x, the estimate from the adjustment is called the least–squares estimate or sometimes the adjusted value, denoted by x . the residual has been defined by equation and will be used in the same sense as it has been in the past. The deviation is simply the negative of the residual and may be used on occasion in the course of statistical computation.

3.3.3.1 SAMPLING

The statistical term population refers to the totality of all possible values of a random variable because of the large size of the population. It is either impossible or totally impractical to seek all of its elements to evaluate its characteristics. Instead, only a certain number of observations, called a sample, is selected and studied.

From the sample data. Some or all of the following can usually be computed:

1. frequency diagrams (histograms and stereograms).
2. sample statistics for location (mean, median, mode , midrange)
3. sample statistics for dispersion (variances and covariance's).

3.3.3.1.1 Histograms and stereograms

A histogram is constructed for a single random variable, but for two random variables, the frequency diagram is called a stereogram.

3.3.3.1.2 SAMPLE STATISTICS FOR LOCATION

a. The sample mean

The first and most commonly used measure of location is the main of a sample, which is defined as

$$\overline{x} = \frac{1}{n}\sum_{i=1}^{n}\left|(xi)\right|$$

where $\overline{x}$ is the mean , x_i are the observations. And n is the sample size. Or total number of observations in the sample.

b. The sample median

Another measure of location is the median, x_m, which is obtained by arranging the values in the sample in their order of magnitude. The median is the value in the middle if the number of observations n is odd and the mean of the middle two values if n is even . thus, the number of observations larger than the median equals the number smaller than the median.

c. The sample mode

The sample mode is the value that occurs most often in the sample. Therefore, if a histogram is developed from the sample data, it is the value at which the highest rectangle is constructed.

d. Midrange

If the observation of smallest magnitude s subtracted from that of the largest magnitude, a value called the range is obtained. The value of the observation that is midway along the range, called the midrange, may be used, although infrequently. As a measure of position for a given sample. It is simply the arithmetic mean of the largest and smallest observations.

3.3.3.1.3 SAMPLE STATISTICS FOR DISPERSION

a. The range

The simplest of dispersion (scatter or spread) measures of the given observations is the range, as defined in the preceding section; however, it is not as useful a measure as others.

b. The mean (or average) deviation

The mean deviation is a more useful measure of dispersion and has been conventionally called the average error, it is the arithmetic mean of the absolute values of the deviations from any measure of position (usually the mean) . thus, the mean deviation from the mean for a sample of n observations would be given by

$$\text{Mean deviation} = \frac{1}{n}\sum_{i=1}^{n}\left|(xi - \overline{x})\right|$$

c. Sample variance and standard deviation

The mean deviation, although useful in certain cases, does not reflect the dispersion or scatter of the measured values as effectively as the standard deviation, which is defined as the positive square root of variance . the variance of a sample is

$$\pm^{2}x = \frac{1}{n-1}\sum_{i=1}^{n}(xi - \overline{x})^{2}$$

where is the sample mean and n the sample size. The sample variance $\pm^{2}x$ is an

unbiased $\overline{x}$ estimate of the population variance $\pm^{2}$. it is often called the estimate of the variance of any one observation. On the other hand, the estimate of the variance of the mean $\overline{x}$ is given by

$$\pm^{2}x = \frac{\pm^{2}x}{n}$$

3.4 MEASURES OF QUALITY

Several terms are used to describe the quality of measurements and the quantities derived from them. The following are those most commonly used in surveying:

Accuracy, the term accuracy refers to the closeness between measurements and their true values, the further a measurement is from its true value, the less accurate it is. (the mathematical terms expectation or expected value are used more often at present than true value.

As opposed to accuracy, the term precision pertains to the closeness to one another of a set of repeated observations of a random variable, if such observation are closely clustered together, then they are said to have been obtained with high precision. It should be apparent, then , that observations may be precise but not accurate, if they are closely grouped together but about a value that is different from the true value by a significant amount. Also, observations may be accurate but not precise if they are well distributed about the true value but dispersed significantly from one another. Finally, observations will be both precise and accurate if they are closely grouped around the expected value (or the distribution mean).

An example often used to demonstrate the difference between the two concepts of accuracy and precision is the grouping of rifle shots on a target.

Different groupings that are possible to obtain. From the preceding discussion, group (a) is both accurate and precise, group (b) is precise but not accurate, and group (c) is accurate but not precise, a hard notion to accept is that case (c) in fact is accurate, even though the scatter between the different shots is rather large. A justification that may help is that we can visualize that the center of mass (which is the true value). The most commonly used measure is the variance or its positive square root, the standard deviation.

Weight. It is easy to see that the higher is the precision , the smaller is the variance. To avoid the apparent reversal in meaning, the term weight is used to express a quantity that is proportional to the reciprocal of the variance (for uncorrelated or independent random variables). It is denoted by 2.

Relative precision. The term relative precision refers to the ratio of the measure of precision, usually the standard deviation, to the quantity measured or estimated. For example , if a distance s is measured with a standard deviation $\pm s.$the relative precision is $\pm s / s$.

Ratio of misclosure, in traverse computations the ratio of misclosur (ROM) is given by

$$ROM = dc/S$$

Where d_c is the miscolosure in m or ft and s is the length of traverse in m or ft. although the ratio of misclosure has been used by some practicing surveyors to classify the quality of traverses perfumed by traditional methods. It has no statistical basis and its use therefore is not recommended.

Relative line accuracy. When the propagated standard deviation, $\pm v$,is given for a line of length S_{ij} , between points i and j . then the relative line accuracy (RLA) is given by

$$RLA = (\pm_{ij}) / (s_{ij})$$

Note that the RLA is just a special example of the general concept of relative precision.

Mean square error. When the so-called true values are available to compare with calculated values. The mean square error (MSE) is given by

$$MSE = \left[\sum (x - ^2)^2\right] / n$$

In which x is the measured value, t is the true value, and n is the number of measurements.

Root mean square positional error. Horizontal positions are frequently specified by x and y rectangular coordinates. The root mean square positional error (RMSPE) is

$$RMSPE = (MSEx + MSEy)^{1/2}$$

In which x the measured value, ^{2} is the true value, and n is the number of measurements.

In which MSEx = 1/n Σ(Xmeas - Xtrue)^2 and MSEy = 1/nΣ(Ymeas – Ytrue)^2 each is compute with equation above. the RMSPE is often used in determining map accuracy.

3.5 ERRORS IN PRIMARY MRTHODS OF DATA COLLECTION

The primary methods of data collection include the techniques of geodesy, photo grammetry, and surveying. All these techniques include random, systematic, and gross errors. However, the methods of observations as well as computation are designed in such a way that gross errors are eliminated, systematic erors are either corrected or are mathematiclly modeled, and random errors are treated systematically by stochastic models, for example, by using the method of least squares.

Errors introduced in the primary method of data collection include the following types:

- Personal errors .
- Instrumental errors .
- Environmental errors .

3.5.1 Personal errors

- occur because no observer (surveyor or geodes its or photogrammetist) has perfect senses if sight and touch .

This type of error includes reading errors, centering errors .as well as bisection errors. This also includes the personal equation which involves how a particular individual estimates the readings between graduations. personal errors could be blunders and systematic as well as random errors .

3.5.2 Instrumental errors

include errors caused by imperfect instrument construction, or lack of adequate instrument adjustment or calibration prior to its use in data collection. Error introduced by instrumental effects are mainly systematic in nature.

3.5.3 Environmental errors

are primarily caused by variations temperature, pressure, humidity , magnetic variations, obstruction of signals, winds, and illumination at the time of observation.

3.6 ERROS IN SECONDARY METHODS OF DATCOLLECTION

The secondary methods of data collection include all the errors contained in the primary methods. In addition the secondary methods incur the following errors :

- error in plotting control .
- Compilation error .
- Error introduced in drawing.
- Error due to map generalization .
- Error in map reproduction .
- Error in colors registration .
- Deformation of the material.
- Error introduced due to the use of wrong scale .
- Uncertainty in the definition of a feature .
- Error due to feature exaggeration.
- Error in digitization or scanning .

3.6.1 ERROR IN PLOTTIG CONTROL

The first step involved in making a map is to plot the control points . The root mean square error (RMSE) involved in this process, e_1, varies between 0.17 and 0.32 mm for coordinatographs attached to photogrammetric plotters. The error introduced during a control survey using for example the global positioning system (GPS) can be ignored at plotting scale cause one achieve centimeter level accuracy.

3.6.2 COMPILATION ERROR

Compilation of topographical maps entails bringing data from various sources to a common scale by photography the error introduced in this process (malign 1989)is e_2 = k e the value of e_2 ranges between 0:30 mm and 0:32 mm where e is the error in the detail survey which ranges from 5 m to 7.5 m if the detail is collected by photogrammetric method for a map with a scale 1:25,000, and k is the amount of reduction in scale from that the plotted detail to that of the compiling manuscript .it should also be noted that in digital mapping point features will have a different accuracy than line features. Normally, well defined point features can be compiled more accurately than line features moreover even within line features the accuracy of compilation will vary depending on the definition and thickness the features.

3.6.3 ERROR INTRODUCED IN DRAWING

The drawing error, e_3, is usually introduced at fair drawing stage and is quoted (Maling 1989)as ranging from 0.06 mm and 0.18 mm.

3.6.4 ERROR DUE TO GENERALIZATION

The generalization error is very difficult to quantify because the amount of error introduced depends on the type of features and also on the character (or complexity) of the feature. The error could range from substantial for some features.

Error due to deliberate features to be portrayed on a map are so close that they cannot be plotted in their proper position without overlapping .Therefore, they are displaced at the time of plotting to make the map legible .for example if there is a road one side of a river and railway line on the other side ,then three features cannot be plotted without displacing some them. The smaller the scale of a map the larger the displacement. Again this error could be substantial depending on map scale and the proximity of the features to be portrayed.

3.6.5 ERROR IN MAP REPRODUCTION

The RMSE in map reproduction, e_5, varies between 0.1 mm and 0.2 mm

3.6.6 Error IN COLOR REGISTRATION

A color map is reproduced from a series of metal printing plates which are used to print on a paper one color at a time. The RMS error introduced in proper registration e_7 varies between 0.17 and 0.30 mm .

3.6.7 ERROR INTRODUCED BY THE DEFORMATION OF THE MATERIAL

Maps are normally printed on paper the dimensions of paper change with changes in humidity and temperature when an increase in humidity, the moisture content of paper may increase from 0 percent to 25 percent with a corresponding change in paper dimensions of as much as 1.6 percent at room temperature. The paper will not return to its original size even if the humidity is reduced because the rates of expansion and shrinkage are not the same. A 36-inch long paper map can change by as much as 0.576 inches due to a change humidity.

Nearly all materials increase in dimension when heated a decrease when cooled paper is no exception to this rule at time of printing the paper temperature is high therefore can be stretched up to 1.5 percent in length and 2.5 width (Maling, 1989) after the paper dries and cools it shrinkage by 0.5 percent in length and 0.75 percent in width. The net change in the dimensions of the paper map after printing and cooling may be 1.25 percent in length and 2.5 percent in width .

3.6.8 ERROR INTRODUCED BY THE USE OF UNIFORM SCALE

The scale quoted in a map is what is known as the principal scale which is true. For example, for the Lambert conformal projection the principal scale is true only along standard parallels the scale is too small between the parallels and too large outside the parallels therefore one should use the proper scale factor correction when digitizing a map or when measuring distances from maps .

When information from different maps is collected then one has to make sure that they are using the same map projections and are of compatible scales. Revised “old” maps may have used different map projections in a new edition but this may not have been stated in the peripheral information.

DEFINITION OF FEATURE

Many natural features do not have a clear cut boundary for example where does one exactly mark a coastline ? Is it at mean high water (MHW), mean higher high water (MHHW), or mean Low water (MLW)? Other features, such as the boundary between forested areas and non forested areas are also fuzzy. The width of a river is different at rainy and dry seasons, etc. in the position of a feature. It must be recognized that not all features will exhibit this error .

3.6.9 ERROR INTRODUCED DUE TO FEATUR EXAGGERATION

In order to increase the communicative value and legibility of a map, features are sometimes exaggerated because they can not be portrayed at their proper dimensions for example a boundary line normally does not have a width yet when it is plotted on a map it occupies a substantial width. Some features are more exaggerated than others depending on the purpose of a map. for example roads are, exaggerated on a road map. Error due to feature exaggeration could be substantial depending on the scale and purpose of the map and the type of feature involved. Again, one must point out that not all features will have this kind of error.

3.6.10 ERROR IN DIGITAZTION

Digitization and scanning errors depend on the following factors :

- Width of the feature.
- Skill of the operator.
- Complexity of the feature.

- Resolution of the digitizer.
- Density of the features.

When digitizing a thick line it is difficult to continually place the curser on the middle of the line .the operator is also likely to make more errors when digitizing in areas where the features are dense for example contour lines in mountainous areas. The operator is also likely to make errors when he/she is tired. Note that errors for point features will not be the same as for linear features the digitization error is quoted as e_{11} = 0.25 mm. Line following techniques and scanners perhaps introduce fewer planimetric errors but errors in feature tagging could be higher in the case of scanning.

3.7 TOTAL ERROR

It is very difficult if not impossible to assess the total error introduced in the secondary methods of data collection because we do not know the functional relationship among the various error introduced at different stages of the mapping processes dimensional instability of the medium, and digitization as summing that a linear relationship exists between the total error and the individual, errors the total error may be computed by using the law of propagation of errors.

Total error $=(e_1^{\,2} + e_2^{\,2} + e_3^{\,2} + e_5^{\,2} + e_6^{\,2} + e_7^{\,2} + e_{11}^{\,2})^{1/2}$

Worst case scenario

Total (RMS) error $= (0.32^2 + 0.32^2 + 0.18^2 + 0.2^2 + 0.30^2 + 0.48^2 + 0.25^2)^{1/2} = 0.18$ mm at map scale.

Best case scenario

Total (RMS) error = $(0.01^2 + 0.30^2 + 0.06^2 + 0.10^2 + 0.17^2 + 0.24^2 + 0.25^2)^{1/2}$ = 0.50 mm at map scale.

The above computations show that the positions could be off by several meters if we use maps at scales of 1:24,000. This obviously is an unacceptable error for many applications.

بسم الله الرحمن الرحيم

تحية الوطن وبعد.

نرجو من حضرتكم التكرم بتعبئة الاستبيان التالي ، حيث يهدف الاستبان الى الحصول على معلومات حول الخرائط المستخدمة في المؤسسات الفلسطينية فيما يتعلق بجودتها ، بهدف البحث العلمي الخالص ،وذلك أستكمالا لمتطلب التخرج في جامعة بوليتكنيك فلسطين .

١. ما هي أنواع الخرائط المستخدمة في مؤسستكم؟
٢. مصادر هذه الخرائط؟
٣. طريقة أعداد هذه الخرائط؟
٤. أهداف هذه الخرائط؟
٥. ما هي المواصفات أو المقاييس التي يتم أستخدامها في هذه الخرئط؟
٦. هل يوجد مواصفات فلسطينية يتم أستخدامها؟
٧. ما هي الفترة التي يتم بعدها تحديث هذه الخرائط؟
٨. الجهة التي تقوم بعملية التحديث؟
٩. ما هي طرق فحص وتقييم الخرائط لديكم؟
١٠.أنواع الاسقاطات المستخدمة في عمل هذه الخرائط؟

ELECTRONIC DISTANCE MEASURMENTS

principle of the electronic measuring devices

These instruments operate by transmitting beams of electronic or micro waves between instruments located at the ends of the line being measured.

The beams are modulated at precisely controlled frequencies and bycomparing the transmitted and received beams at one or both ends of the line ,the distance of the line can beaccurately computed. As the destances computed with thrvelocity of the electromagnetic radeateon,its accuratvelocity is required to be known,along with necessary corrections of tempera ture ,atmospheric pressure and relative humidity .the end stateons of the line must beintervisible. The distance odtained by these instruments is the slope length of the line .

classification of e.d.m lnstruments

EDM . instruments for surveying may be divided into twoclasses :

(1) the instruments in which a beam of light is used as the carrier and which gets reflected back from a mirror located at the other end .

such instruments are less expanseve because one active instrument and one battery only are needed at one end and the instrument at the other end is simply a reflecteng mirror centred over rhe ground station mark.

Such instruments are used only for short distances. When the destance between the end stateons is comparatibvely large, a telescope is used for aiming the bean at the nirror and laser is used as carrier because this has a very small angle of spread.

(2) the instruments which transmit short radio waves (micro waves)as carrier .ln thes type of instruments,an active instrument and as operator are needed at either end of the line .the signal is retransmitted on another carrier wave back to the originating instrumint. The wavelengths of the carreer of such instruments varies from 8 mmto 10 mm these instruments do not require precise directronalsetting because the spresd angle of the carrier waves is between 5to 10 . lndtruments using microwaves are generally used for long distances required in primary triangulation and precise traversing .

Trilateration:

The type of the triangulation in which the sides of the triangles instead of the angles are measured ,is called trilateration .

In trilateration ,the length of the sides of triangles are determined with the help of electronic distance measuring devices .

The three angles of the triangle are computed trigonometrically from its observed sides .

Knowing the calculated sides ,a starting azimuth and known coordinates of a starting station , the coordinates of remaining stations can be computed .

Measurement of the sides :

After deciding the layout of the triangles ,direct distances or slope distance between trilateration station ,are observed with electronic distance measuring instruments .

<u>The correction to be applied to the observed distance :</u>

To reduce the observed slope distance to M.S.L arc distance the following corrections are applid :

1. Instrument constants .
2. Atmospheric correction .
3. Slope correction .
4. Chord to arc correction .
5. M.S.L correction .

❖ Istrument constants :

The value of additive constant of EDM instrument may be determined as under :

A--------X1-----------------B------------X2----------------C

1. Measure a base line accurately up a millimetre .
2. Divide the base line into suitable segments and measure their lengths.

Let AB=X1 and BC=X2 .

Where n is the number of segments of the base

EXAMPLE 9.1. THE GROUND DISTANCES of two segments AB and Bcof a base line are 606 .458 m and 661.935 m respectively.their measured distances with an E.D.M. instrument are 606 .612 m and 652.075m and total distance AC is 1268.545m calculate the additive constant of the E.D.M.instrument .

2.refracteve lndex and atmospheric correcteon .the ratio of the speeds of the measuring signal in the vacuum and through air at the time of measurement is called the refracteve index .

Let n0=refractive index along the E.D.M.ray path at the time of observation .ln ordinary air ,its value is taken as 1.0003.

ELECTRONIC DISTANCE MEASURDMENTS

Ns =refractive index along the E.D.M. ray path at the time of standardisation .

D = slope distance for ranging then

A tmospheric corrextion

Catmosp = (n0 – Ns) D.

the exact value of refractive indexdepends on the pressure ,temperature and humiditiy od the atmosphere .hence, the meteoro logical readings for pressure and humedety are taken at the time of the observation simultaneously at either end ot of the line .the number of parts per million by which the speed of electromagnetic waves in air differs from that in vacuum is (n0 – 1)*10 6.

Since the introduction of EDM,there has been much research towards finding the appropreate formulae for calculating the refractive index by making meteorological observations at the time of observation .some of the well adopted formulae are as under .

(1)froome and essen formula (for EDM using radio waves)

(n0 – 1)106 =103.49 (P-E)+86.26 (1+5748)

T T T

Where p= total atmospheric pressure in millibaes of mercury barometer

E =partial pressure of the water vapours .lts value is obtained with the helpof a hygrometer wet bulb and dry bulb thermometers .

T = absolute temperature i.e.(t0 +273). Substituting the value of T =300,p=760,e=10, in eqn .

(9.2) we get

(Ns –1)106=103.49 (760-10) + 86.26 (1+5748)*10

300 300 300

=103.49 750 +86.26*6048 *10

300 9000

Ns= 1*(103.49 75+86.266048)

106

set up the instrument at the ends of each segment and measure their distances with an EDM instrument .

let e.d.m distance ,ab=y1

let e.d.m distance bc=y2

let e.d.m distance ac=y

(4)calculate the additive constant as under :

k=ac-(ab+bc)

$$= y-\{(y1+y2+.......+yn \over n-1$$

where n is the number of segments of the base

example 9.1. the ground distances of two segments ab and bc of a base line are 606.458 m and 661.935m resoectively .their measured distances with an e.d.m .instrument are 606.612m and 652.075 m and total distance ac is 1268.545m

calculate the additive constant of the e.d.m instrument

solution

---------------------- x =1268.393m--------------------------

x1=606.458m

0----------------------------0-----------------------------0

ay1=606.612m B y2=662.075 C

-----------------------y=1268.545m--------------------------

we know that additive constant

$$k = \frac{y-(y1+y2)}{n-1}$$

substituting the values in eq (i)wi get

$$k = \frac{1268.545-(606.612+662.075)}{2-1}$$

$$= \frac{1268.545-1268.687}{1}$$

= -0.142ans

2.refractive lndex abd atmospheric correction .the ratio of the speeds of the measuring signal in the vacum and through air at the time of measurement is called the refractive index

let n0= refractive index along the e.d.m ray path at the time of observation .ln ordinary air its value is taken as 1.0003 .

ELECTRONIC DISTANCE MEASUREMENTS

Ns =refractive index along the e.d.m ray path at the time of standardisation

D= slope distance for ranging ,then

Atmospheric correction

C atmosp =(n0-Ns)D

The exact value of refractove index depends on the pressure temperature and humidity of the atmosphere.

Hence, the meteoro logical reading for pressire and humidity are taken at the time of the observation simultaneously at either end of the line .

The number of parts per million by which the speed of electromagnetic waves in air differs from that in vacuum is (n0-1) 106.

Since the introduction of EDM, there has been much research towards finding the appropriate formula for calculating the refractive index by making meteorological observations at the time of observation .

Some of the well adopted formula are as under .

1. Froome and Essen formula (For EDM using radio waves) .

Where:

P=total atmospheric pressure in millibars of mercury barometer .

E=partial pressure of the water vapours .Its value is obtained with the help of a hygrometer wet bulb and dry bulb thermometer .

T=absolute temperature .

2. Barrel and Sears formula (for EDM using light beam carrier)

Where :

λ= the wavelength in micrometres .

In this formula, the only term (-15.02E/T) depends on the atmospheric humidity and is usually ignored, the value being too small .

3. I.U.C.G formula (for EDM using micro waves)

$$(N_s-1)10^6=\frac{77.624\ p}{T}\left(\frac{12.924}{T}\right)+(3.719 * 10^5 .\frac{e}{T^2}$$

Where $e=e-(0.00066+759 * 10^{-9}\ t)(t-t)p$

E=saturation vapour pressure in millibars for t over ice /water in millibars

T= t+273

T =dry bulb temperature

T =wet bulb temperature

P = pressure in millibars by an electronically operated barometer correct up to 0.1 millibar

As the refractive index is a function of p ,t and t an error in obser vation of these parameters, affect the measured distances the standard refactive index for air microwaves at 0C temperature and 760 mm pressure ,is 1.000325.

Corrected slope distance for refractive index

$$Dri = \frac{NS.dM}{N^o}$$

Where Drt= corrected slope distance for refra

Field and office work

General

The nature of surveying measurements already has been indicated .much of the fled and office work involved in the acquisition and processing of measurement is performed concur rently .field and office work for a complete survey consists of

1) Planning and design of the survey ,adoption of specifications ,adoption of a map projec tion and coordinate system and of a proper datum, selection of equipment and proce dures .
2) Care ,handling ,and adjustment of the instruments
3) Fixing the horizontal lovation of objects or points by horizonta; angles and distances by one of the surveying procedures to be developed
4) Determining the elevations of objects or points by one of the methods of leveling.
5) Recording field measurements .

6) Field computations to verify the data .

7) Office computations on which data are reduced ,adjusted ,and filed or stored for current or future use .

8) Setting points in the field to dispalay land property locateon and to control construction layout (as may be necessary).

9) Perdorming the final as built surviy in which all structures built as a part of the project are located with respect to the basic control network or established property lines .

Discussions in this chapter are restrocted to general comments related to planning and designing the survey ,care and adjustment of instruments ,methods for recording data, computional methods ,and computational aids .

PLANNING AND DESIGN OF THE SURVEY

There are many types of surveys related to an almost infinite varety of projects .therefore the planning of the survey can be discussed only in very general terms at this point .assuming that the nature of the project is established and the results desired from the survey are known the steps involved in planning the survey are these :

1) Establish specifications for horizontal and vertical control accuracies

2) Locatr and analyze all existing control ,maps photograpgs and other survey data

3) Do a preliminary examination of the office using existing maps and photo graphs and in the field to locate existing and set new control points

4) Select the equipment and surveying procedures appropriate for the task

5) Select the computational procedures and the method to present the data in a finsl form

To understand the procedure ,consider an example .assumple that a priate surveying firm gas been engaged to prepare a topographic map of a 2000 acre (809 .4ha)tract at a scale of 1:1500 (see swction 1.9 for a definition of scale)and with a contour interval of 2m .* control surveys are to be performed using a combination of theodolite and EDM traverse (see section 8.38 chapter 8)and the global positioning system (GPS ,see chap ter 12)the major portion of the topographic mapping is to be compiled photogrammeti cally the tasks to be performed by the surveying firm are to selwct a prorer coordinate grid and datum plane and to establish a horizontal and vertical control network throughout the area including sufficient points to control aerial photographs for the photogrammetric map ping and to map heavily wooded valleys not accessible to the photogrammetric methods

Another firm has been engaged to do thephotogrammetric work

The map is to be used by the client for presenting plans of a proposed industrial park to the county planning commission .lf plans are approved earthwork quantities will be calculated using measurements from the map and construction surveys will be made from control points set in the survey consider some of the problems involved in planning for this type of a survey .

SPECIFICATIONS

The accuracy of the measurements should be consestent with the purpose of the survey each survey is a problem in itself ,for wjoch the surveyor must establish the limits of error using a knowledge of the equipment involved the procedure to be employed error propaga tion and judgment based on practecal experience the best survey is one that provides data of adequateaccuracy without wasting time or money the federal geodetic control subcommittee (fgcs)publishes specifications for first second and third order horizontal and vertical control surveys and for the gps relative positioning techniques (see chapter 5and 9)these specifications provide a starting point for establesging standards on most jobs that require basic control surveys .

For the example given the work can be divided into three categories establishing basic control establishing supplementary control and performing topographic aurveys .

Basic control consists of a primary network of rather widely spaced horizontal and vertical control points that will coordenate with local state and national controls and be used to control all other surveys throuhgout the entire area to be studied second order specifications generally are recommended for basic control networks for this phase GPS methods most often are used because photogrammetric surveys will follow particular attention should be paid to establihing basic horizontal control points around the perimeter of thr area to be mapped plus several vertical control points near thr center of the project constuction surveys are definitely a possebiliy so that a sufficient number of basic control points sgould be established in the interior of the area to allow tying all subsequent construc tion surveys to secoond order control points .

Supplementary control is set to provide additional control points for photogrammetric mapping and for topographic mapping by field methods therd order specifecations or lower (down to a ratio of misclosure (ROM see section 2.11 chapter 2)of l part in 3000)usually are consedered adequate for supplementary horizontal control total station system tra are comsedered adequate for supplementary horizontal control total station system tra verses generally are used for this category of syrvey .on the example project most of thesupplementary control will be used for controlling the photogrammetric mapping and a specification for error in position is appropriate therefore it could be specidied that supplementary control points should be to within ± a tolerance vased on the requirments of the photogrammetric mapping system vertical control usually is estsblished to astisfy third order specifications close cooperation is required between the survyor and pho togtammetrisst in this phase of the work under certain circumstances supplementary control can be established by photogrammetric methods which could be a topic for discus sion between the surveyor and the photogrammetrist topographic measurents in thefield are made from supplementary control points accuracies in these measurements usuallu are based on map accuracies for the example if the map scale is 1:1500then 0.5 mm on the map is equivalent to 0.75m on the ground or 1 in represents 2.5ft thus thr accuracy

50

in position formapping is ± 0.8 m or ± 2.5 ft elevations are governed by the contour interval national map accuracy standards (sec tion 14.26)erquire that 90 percent of all spot elevations detetrmined from the map be within ± one half contour interval of the correct elevation experience has shown that this accu racy of contour interpolation normally can be achieved for maps if spot elevations in the field are determined to within one tinth of the contour interval heights of topograohic points should be located to within ±0.2 m for the example project because elevations within this interval can be well determined by both field and photogrammetric survey method .

the specifications for photogrammetric mapping are largely a function of the map scale and contour interval as thr contour interval decreases the cost of the photogrammetric map increases at an exponential rate therfore it is worth studying the desired contour interval to be sure that this interval is really necessary for this example earthwork will becalculated from measurements made on the map so that both contour interval and map scale represent optimum values that must be maintained thes being the case aerial photog raphy must be flown at an altitude anf to specifications compatible with the stated scale and contour interval

unique situations on the project may require special treatment lf construction is to be concentrated in a particular area of thr site second order control can be clustrered in that region and mappping of that portion can be at a larger scale and with a smaller contour interval for a relatively small area such as the example problem of 2000 acres mapping at two scales probably would not be practical such an approach could be feasible under special circumatances in larger areas the requrements for construction surviys maybe the limiting factor when strucures for water or other liquids by gravity flow are invikved furst irder keveks are required bridges complicated steel structures tunnels and city srveys are a few examples where first order control surveys may be necessary

EXISTING MAPS AND CONTROL POINTS

A thorough searcg sgiykd be oerfirned tokicate akk survey data in all existing giographec or land lnformation systems or other data bank sources existing maps and aerial photo graphs of the area to be surveyed. The national gEodetic surveiy division (ngsd)of the national ocean and atmo apheric administration (noaa)in silver springs maruland 209 10.maintains records of all first second therd and selected fourth second third and selected fouth orrder and photogrammetrically derived con trol point locations in the horizontal and vertical control networks of the united states control point data plus descriptions will be property survey data related to publec lands survey state county city and town engineereng and surveyor “s offices also shuld be consulted for useful survey data

SELWCTLON OF EQULPMENT AND PROCEDURES

The specification for horizontal and vertical control provide an allowable tolerance for horizontal and vertical positions .using estimates for the capadilities inherent in various types of instruments errors in distance and direction can be propagated for a given proce dure these propagated errors then are compared with the allowable tolerance and the instrument or proceduer can be modified if necessary various instruments procedures for use of these instruments and error propagation methods are described in subsequent chapters considerable judgment and practical experience in surveying are required in this phase of the survey

SELECTION OF COMPUTATIONAL PROCEDURES AND THE METHOD FOR DATAPRESENTATION

Systematic procedures for gathering filing processing and disseminating of the survey data should be developed field notes and methods for recording data are described in section 3.12

The prevalence of modern efficient electronic computing equipmint virtually dictates that rigorous methods and adjustment techniques be utilized in all but the smallest of surveys

Methods for filing or storing survey information should be standardized for all jobs and ought to allow current or future reteieval with no complications the use of a giographic or land lnformation systim (GIS or LIS)that has an interface with a computer aided drafting and design (CADD)system (see chapter 14)is highly recommended as an effi cient way to exploit current electronic technology for storing and filing data

The method for presenting the data must be carefully consedered for thr examole case a conventional line map or an orthopgoto map (see chaoter 14)with an overlay of contours would be appropriate for presentation to a planning commission selection of the method for presenting the data underscores the value of using GLS or LIS and CADD for example if a line map wirth an overlay of contours were chosen then most sueveying firms with GIS /CADD capability would be able to provide this product quickly at a minimum cost

RELATION BETWEEN ANGLES AND DLSTANCES

Ln section 3.5 refernce was made to the propagated errors in distance and en direction these erro rs are used to betermine the uncertainty in position for a ponint assume that esti mated standard deviations in a distance r and direction a are and respectively as de veloped in section 2.31 and example 2.15 the position of a point determined using r and a has an uncertainty region defined by an ellipse centered about a point located by r and a as shown in figure 3.1 ln example 2.15 =0.5m.r =100m.a =60and =30 which produce an ellipse having semiminor axis of =0.5 mthat is parallel to r the semimajor axis of the ellipse =r=0.87 mand is nornal to line r note note that =-0.61 m and =0.79 m both of which are less than the maximum uncertainty in the point =r=0.87 m

Suppose that is chosen so that r=r or =0.5/100 =0.00500 rad ln this case the two axes of the rehion of unceraínty are equal and the elliose becomes a circle as illustrated in figure 3.2 therefore to have the same contribution fron distance and angle errors should be about 0 17

The preceding anaysis illustrates the relationship between uncertainties in direction and distance and emphasizes the desirability of maintaining consistent accuracu the two measuremints the error in distance is normally expressed as a relative preceseon of ratio

Of the error to the distance (see section 2.11)ln the examole the relative precision is 0.5/100or l part in 200 similarly thr linear distance subtended by in a destance r equals 0.5 and thr tangent or sine of the error or its value in radians is l part in 200 accordingly a consistint relation beween accuracies in angles and distances will be maintained if the estimated standard deviation in direction equars radians or the relative precision in the distance

Lt is impossible to maintain an exact equality between these two relative accuaceces but with some exmeptions to be consedered presently surveys should be conducted so that the difference between angular and distance accuracies is not great table 3.1 shiws for various angular standard deviations the corresponding relative preceseon and the linear errors for lengths of 1000ft and 300 mfor a length other than 1000ft or 300 mthe linear error is in direct proportion a convenient relation to remember is that an angular error of 01 corresponds to a lineear error of about 0.3 ft in 1000ft or 3 cm in 100m

To illustrate the use of the table suppose that distances are to be measured with a precision of 1/10,000 from the table the correspondeng permissible angular error is 20 as another examole suppose that the distance from the instrument to a desired point is determined as 250m with a standard deviation of 0.8 mfor an angular error of 10 the

Corresponding linear error is (250/300)(0.87)=0.73m .therefore the angle needs to be determined only to the nearest 10. the prevalence of electronic disance measurement (edm)equipment for measuring distances creates a situation where an exception occurs distances can be measured using edm with a very good relative preciseon without additional effort for example suppose that a destance of 3000 m is observed with an estimated standard deviation of 0.015 m ,producing a relative precision of 1/200,000 .the corresponding angular standard deviation is 01.for an ordinary survey this degree of angular accuracy would be entirely unneces sary at the other end of the spectrum distances may be determined roughly by taping or pacing and angles measured with more than the required accuracy for example in rough taping the relative precision in distance might be 1/1000 corresponding to a standard deviation in angles of 03 even using an ordinary 01 transit angles could be observed as easily to the nearest 01 as to the nearest 03

Often field measurements are made on the basis of computations involving the trigonometric functions and it is necessary that the computed results be ofa requir ed precision lf the values of these functions were exactly proporyonal to the size of the angles in other words if any increase in the size of an angle were accompanied by a proportional increase or decrease in the value of a function the problem of determin ing the precision of angular measurements would resolve itself into that explained in the preceding section however because the rates of the sines of small angles of the cosines of angles near 90 and the tangents and cotangints of small and large angles are relatively large it is evident that the degree of precision which an angle is determined should be made to depend on the size of the angle and the function to be used in the computations lt is not practical to measure each angle with exactly the precis ion necessary to ensure suddiciently accurate computed values but at least the surveyor should have a sufficiently comprehensive knowledgi of the purpose of the survey and the properties of the trigonometric functions to keep the angles within the required preci sion

The curves of figures 3.3 and 3,4 show the relative precision corresponding to various standard deviations in angles from 05 to 01 for sines cosines tangents and cotangents for the function under consideration these curves may be used as foiiows :

1) To determine the relative precision corresponding to a given angle and error

2) To determine the maximum or minimum angle that for a given angular error will furnish the required relative precision

3) To determine the precision with which angles of a given size must be measured to maintain a required relative precision in computations

CHAPTER 6

AGENCIES SPESIFICATIONS

LAND AND GEOGRAPHIC INFORMATION SYSTEMS

Land information systems (LISs) and geographic information systems (GISs) are new areas of activity which have rapidly assumed positions of major prominence in surveying. These computer-based systems enable storing, integrating, manipulating, analyzing, and displaying virtually any type of spatially related information about our environment. LISs are being used at all levels of government, and by businesses, private industry, and public utilities to assist in management and decision making. Specific applications have occurred in many diverse areas and include natural resource management, facilities sitting and management, land records modernization, demographic and market analysis, emergency response and fleet operations, infrastructure management, and regional, national, and global environmental monitoring.

Data stored within LISs and GISs may be both natural and cultural, and be derived from surveys, existing maps, charts, aerial and satellite photos, statistics, tabular data, and other documents. Specific types of information, or so-called layers, may include political boundaries, individual property ownership, population distribution, locations of natural resources. Transportation networks , utilities, zoning, hydrograph, soil types, land use, vegetation types, wetlands, and many more.

An essential ingredient of all information entered into LIS and GIS databases is that it be spatially related, that is, located in a common geographic reference framework. only then are the different layers of information physically relatable so they can be analyzed using computers to support decision making. This geographic positional requirement will place a heavy demand upon surveyors in the future, who will play key roles in designing, implementing, and managing these systems. Surveyors from virtually all of the specialized areas described in the preceding section will be involved in developing the needed databases. Their work will include establishing the required basic control frameworks, conducting boundary surveys and preparing legal descriptions of property ownership; performing topographic and hydrographic surveys by ground, aerial, and satellite methods; compiling and digitizing maps; and assembling a variety of other digital data files.

This subject seems appropriately covered at the end, after each of the other types of surveys needed to support these systems has been discussed.

FEDERAL SURVEYING AGENCIES

Several agencies of the U.S. government perform extensive surveying and mapping. Four of the major ones are:

1. The coast and Geodetic survey. Now the national geodetic survey (NGS) and part of the national ocean survey (NOS). was originally organized to map the coast. Its activities include control surveys, preparation of nautical and aeronautical charts, photogrammetric surveys, tide and current studies , collection of magnetic data, gravimetric surveys, and work wide control survey operations that involve satellites. The basic control points established by this organization are the foundation for all large area surveying. The NGS also plays a major role in coordinating and assisting in activities related to the development of modern LISs at local, state, and national levels.

2. The Bureau of land management (BLM) originally established in 1812 as the general land office, directs the public lands surveys. Lines and corners have been set for most public lands in the conterminous united states, but much work remains in Alaska and is proceeding with modern techniques.

3. The U.S Geological survey (USGS). Established in 1879, has the responsibility for preparing maps of the entire country. Its standard 7 ½ quadrangle maps show topographic and cultural features, and are suitable for general use as well as a variety of engineering and scientific purposes. Nearly 10 million copies are distributed each year. Currently the USGS is engaged in a comprehensive program to develop a national digital cartographic database, which will consist of map data in a computer- readable format.

4. The defense Mapping Agency (DMA) prepares maps and associated products, and provides services for the Department of defense and all land combat forces, it is divided into the following military mapping groups: aerospace center, defense mapping school, hydrographic center, inter-American geodetic survey, and topographic center. The DMA topographic center fulfills a key mission in an era when accurate mapping, charting, and geodesy products are essential to realize the complete potential of new weapons. Technological advances in weaponry demand corresponding improvements in mapping, charting, and geodesy to obtain accuracies that were just dreams only a few years ago.

5. In addition to these four agencies, units of the corps of engineers, U.S army, have made extensive surveys for emergency and military purposes. Some of these surveys provide data for engineering projects, such as those connected with flood control. Extensive surveys have also been conducted for special purposes by nearly 40 other federal agencies, including the forest service, national park service, international boundary commission, bureau of reclamation, Tennessee valley authority, Mississippi river commission, U.S lake survey, and department of transportation. Like wise, many cities, counties, and states have had extensive surveying programs, as have various utilities.

THE SURVEYING PROFESSION

Land or boundary surveying is classified as a learned profession because the modern practitioner needs a wide background of technical training and experience and must exercise a considerable amount of independent judgment. Registered (licensed) professional surveyors must have a thorough knowledge of mathematics-particularly geometry and trigonometry, but also calculus: competence with computers: solid understanding of surveying theory, instruments, and methods in the areas of geodesy, photogrammetry, remote sensing, and cartography; some competence in economics (including office management), geography, geology, astronomy, and dendrology: and a familiarity with laws pertaining to land and boundaries. They should be knowledgeable in both field operations and office computations. Above all, they are governed by a professional code of ethics, and are expected to charge reasonable fees for their work.

The personal qualifications of surveyors are as important as their technical ability in dealing with the public. They must be patient and tactful with clients and their sometimes hostile neighbors, few people are aware of the painstaking research of old records required before field work is started. Diligent, time-consuming effort may be needed to locate corers on nearby tracts for checking purposes as well as to find corners for the property in question.

Permission to trespass on private property or to cut obstructing tree branches and shrubbery must be obtained through a proper approach. Such privileges are not conveyed by a surveying license or by employment in a state highway department (but a court order can be secured if a landowner objects to necessary surveys).

All 50 states, Guam, and Puerto Rico have registration laws for professional surveyors and engineers (as do the provinces of Canada). Some states presently have separate licensing boards for surveyors. In general , a surveyors license is required to make property surveys, but not for construction, topographic, or route work, unless boundary corners are set.

To qualify for registration as either a professional land surveyor (LS) or professional engineer (PE) it is necessary to nave an appropriate college degree, although some states allow relevant experience in lieu of formal education. In addition, candidates must acquire two or more years of additional practical experience, and also pass a two day comprehensive written examination. In most states a common national examination covering fundamentals and principles and practice lf land surveying is now used. However, two hours of the exam are devoted to local legal customs and aspects. Thus transfer of registration from one state to another has become easier.

Some states also require continuing education units (CEUs) for registration renewal, and many more are considering legislation that would ass this requirement.

1.introduction

The Government of the United States makes nationwide surveys, maps, and charts of various kinds. These are necessary to support the conduct of public business at all levels of government, for planning and carrying out national and local projects, the development and utilization of natural resources, national defense, land management, and monitoring crustal motion. Requirements for geodetic control surveys are most critical where intense development is taking place, particularly offshore areas, where surveys are used in the exploration and development of natural resources, and in delineation of state and international boundaries.
State and local governments and industry regularly cooperate in various parts of the total surveying and mapping program. In surveying and mapping large areas, it is first necessary to establish frameworks of horizontal, vertical, and gravity control. These provide a common basis for all surveying and mapping operations to ensure a coherent product. A reference system, or datum, is the set of numerical quantities that serves as a common basis. Three National Geodetic Control Networks have been created by the Government to provide the datums. It is the responsibility of the National Geodetic Survey (NGS) to actively maintain the National Geodetic Control Networks **(appendix A).**
These control networks consist of stable, identifiable points tied together by extremely accurate observations. From these observations, datum values (coordinates or gravity) are computed and published. These datum values provide the common basis that is so important to surveying and mapping activities

As stated, the United States maintains three control networks. A horizontal network provides geodetic latitudes and longitudes in the North American Datum reference system; a vertical network furnishes elevations in the National Geodetic Vertical Datum reference system; and a gravity network supplies gravity values in the U.S. absolute gravity reference system. A given station may be a control point in one, two, or all three control networks.

It is not feasible for all points in the control networks to be of the highest possible accuracy. Different levels of accuracy are referred to as the “order” of a point. Orders are often subdivided further by a “class” designation. Datum values for a station are assigned an order (and class) based upon the appropriate classification standard for each of the three control networks. Horizontal and vertical standards are defined in reasonable conformance with past practice. The recent development of highly accurate absolute gravity instrumentation now allows a gravity reference standard.

In the section on "Standards," the classification standards for each of the control networks are described**,** sample computations performed, and monumentation requirements given.

Control networks can be produced only by making very accurate measurements which are referred to identifiable control points The combination of survey design, instrumentation, calibration procedures, observational techniques, and data reduction methods is known as a measurement system. **The section on "Specifications**" describes important components and states permissible tolerances for a variety of measurement systems.

Clearly, the control networks would be of little use if the datum values were not published. **The section entitled "Information**" describes the various products and formats of available geodetic data.

Upon request, the National Geodetic Survey will accept data submitted in the correct formats with the proper supporting documentation **(appendix C)** for incorporation into the national networks When a survey is submitted for inclusion into the national networks, the survey measurements are processed in a quality control procedure that leads to their classification of accuracy and storage in the National Geodetic Survey data base. **To fully explain the process we shall trace a survey from the planning stage to admission into the data base. This example will provide an overview of the standards and specifications, and bow they work together.**

The user should first compare the distribution and accuracy of current geodetic control with both immediate and long-term needs. From this basis, requirements for the extent and accuracy of the planned survey are determined. The classification standards of the control networks will help in this formulation. Here after, the requirements for the accuracy of the planned survey will be referred to as the "intended accuracy" of the survey. A measurement system is then chosen, based on various factors such as: distribution and accuracy of present control, region of the country-, extent, distribution, and accuracy of the desired control; terrain and accessibility of control; and economic factors.

Upon selection of the measurement system, a survey design can be started. The design will be strongly dependent upon the “Network Geometry” specifications for that measurement system. Of particular importance is the requirement to connect to previously established control points. If this is not done, then the survey cannot be placed on the national datum. An adequate number of existing control point connections are often required in the specifications in order to ensure strong network geometry for other users of the control, and to provide several closure checks to help measure accuracy. NGS can certify the results of a survey only if it is connected to the national network.

Situations will arise where one cannot, or prefers not to, conform to the specifications. NGS may downgrade the classification of a survey based upon failure to adhere to the measurement system specifications if the departure degrades the precision, accuracy, or utility of the survey. On the other hand, if specification requirements for the desired level of accuracy are exceeded, it may be possible to upgrade a survey to a higher classification.

Depending upon circumstances, one may wish to go into the field to recover old control and perform reconnaissance and site inspection for the new survey. Monumentation may be performed at this stage. Instruments should be checked to conform to the **“Instrumentation” specifications**, and to meet the “**Calibration Procedures” specifications. Frequent calibration** is an excellent method to help ensure accurate surveys.

In the field, the “**Field Procedures” specifications** are used to guide the methods for taking survey measurements. It must be stressed that the “**Field Procedures” section** is not an exhaustive account of how to perform observations. Reference should be made also to the appropriate manuals of observation methods and instruments.

Computational checks can be found in the **“Field Procedures**” as well as in the “**Office Procedures” specifications**, since one will probably want to perform some of the computations in the field to detect blunders. It is not necessary for the user to do the computations described in the “**Office Procedures” specifications**, since they will be done by NGS. However, it is certainly in the interest of the user to compute these checks before leaving the field, in case **reobservations** are necessary. With the tremendous increase in programmable calculator and small computer technology, any of the computations in the **“Office Procedures” specifications** could be done with ease in the field.

At this point the survey measurements have been collected, together with the new description and recovery notes of the stations in the new survey. They are then placed into the formats specified in the Federal Geodetic Control Committee (FGCC) publications Input Formats and Specifications of the National Geodetic Survey Data Base. Further details of this process can be found in **(appendix C)**, “Procedures for Submitting Data to the National Geodetic Survey.

The data supporting documentation, after being received at NGS, are processed through a quality control procedure to make sure that all users may place confidence in the new survey points. First, the data and documentation are examined for compliance with the measurement system specifications for the intended accuracy of the new survey. Then office computations are performed, including a minimally constrained least squares adjustment. (See appendix B for details.) From this adjustment, accuracy measures can be computed by error propagation. The accuracy classification thus computed is called the “provisional accuracy” of the survey.

The provisional accuracy is compared to the intended accuracy. The difference indicates the departure of the accuracy of the survey from the specifications. If the difference is small, the intended accuracy has precedence because a possible shift in classification is not warranted. However, if the difference is substantial, the provisional accuracy will supersede the intended accuracy, either as a downgrade or an upgrade.

As the final step in the quality control procedure, the variance factor ratio computation using established control, as explained in the section on “Standards,” is determined for the new survey. If this result meets the criteria stated there, then the survey is classified in accordance with the provisional accuracy (or intended accuracy, whichever has precedence).

Cases arise where the variance factor ratio is significantly larger than expected. Then the-control network is at fault, or the new survey is subject to some unmodeled error source which degrades its accuracy. Both the established control measurements and the new survey measurements will be scrutinized by NGS to determine the source of the problem. In difficult cases, NGS may make diagnostic measurements in the field.

Upon completion of the quality control check, the survey measurements and datum values are placed into the data base. They become immediately available for electronic retrieval, and will be distributed in the next publication cycle by the National Geodetic Information Branch ofNGS.

A final remark bears on the relationship between the classification standards and measurement system specifications. Specifications are combinations of rules of thumb and studies of error propagation, based upon experience, of how to best achieve a desired level of quality. Unfortunately, there is no guarantee that a particular standard will be met if the associated specifications are followed. However, the situation is ameliorated by a safety factor of two incorporated in the standards and specifications. Because of this safety factor, it is possible that one may fail to meet the specifications and still satisfy the desired standard. This is why the geodetic control is not automatically downgraded when one does not adhere to the specifications. Slight departures from the specifications can be accommodated. In practice, one should always strive to meet the measurement system specifications when extending a National Geodetic Control Network.

2.standards

The classification standards of the National Geodetic Control Networks are based on accuracy. This means that when control points in a particular survey are classified, they are certified as having datum values consistent with all other points in the network, not merely those within that particular survey. It is n~ observation closures within a survey which are used to classify control points, but the ability of that survey to duplicate already established control values. This comparison takes into account models of crustal motion, refraction, and any other systematic effects known to influence the survey measurements.

The NGS procedure leading to classification covers four steps:

1. The survey measurements, field records, sketches, and other documentation are examined to verify compliance with the specifications for the intended accuracy of the survey. This examination may lead to a modification of the intended accuracy.

2. Results of a minimally constrained least squares adjustment of the survey measurements are examined to ensure correct weighting of the observations and freedom from blunders.

3. Accuracy measures computed by random error propagation determine the provisional accuracy. If the provisional accuracy is substantially different from the intended accuracy of the survey, then the provisional accuracy supersedes the intended accuracy.

4. A variance factor ratio for the new survey combined with network data is computed by the Iterated Almost

Unbiased Estimator (IAUE) method **(appendix B).** If the variance factor ratio is reasonably close to L0 (typically less than1.5*)*, then the survey is considered to check with the network, and the survey is classified with the provisional (or intended) accuracy. If the variance factor ratio is much greater than 1.0 (typically 1.5 or greater), then the survey is considered to not check with the network, and both the survey and network measurements will be scrutinized for the source of the problem.

2.1 Horizontal Control Network Standards

When a horizontal control point is classified with a particular order and class, NGS certifies that the geodetic latitude and longitude of that control point bear a

relation of specific accuracy to the coordinates of all other points in the horizontal control network. This relation is expressed as a distance accuracy, La. A distance accuracy is the ratio of the relative positional error of a pair of control points to the horizontal separation of those points.

Table 2.1—Distance accuracy standards

Classifications	Minimum distance accuracy
First-order	1:100,000
Second-order, class I	1:50,000
Second-order, class 11	1:20,000
Third-order,classl	1:10,000
Third-order, class II	1: 5,000

A distance accuracy, 1:a, is computed from a minimally constrained, correctly weighted, least squares adjustment by

$$a-d/s$$

where

a-distance accuracy denominator.

s-propagated standard deviation of distance between survey points obtained from the least squares adjustment

d-distance between survey points.

The distance accuracy pertains to all pairs of points (but in practice is computed for a sampling of pairs of points). The worst distance accuracy (smallest denominator) is taken as the provisional accuracy. If this is substantially larger or smaller than the intended accuracy, then the provisional accuracy takes precedence.

As a test for systematic errors, the variance factor ratio of the new survey is computed by the Iterated Almost Unbiased Estimator (IALJE) method described in **appendix B**. This computation combines the new survey measurements with existing network data, which are assumed to be correctly weighted and free of systematic error. If the variance factor ratio is substantially greater than unity then the survey does not check with the network. and both the survey and the network data will be examined by NGS.

Computer simulations performed by NGS have shown that a variance factor ratio greater than 1.5 typically indicates systematic errors between the survey and the network. Setting a cutoff value higher than this could allow undetected systematic error to propagate into the national network. On the other hand, a higher cutoff value might be considered if the survey has only a small number of connections to the network, because this circumstance would tend to increase the variance factor ratio.

In some situations, a survey has been designed in which different sections provide different orders of control. For these multi-order surveys, the computed distance accuracy denominators should be grouped into sets appropriate to the different parts of the survey. Then, the smallest value of a in each set is used to classify the control points of that portion, as discussed above. If there are sufficient connections to the network, several variance factor ratios, one for each section of the survey, should be computed.

Horizontal Example

Suppose a survey with an intended accuracy of first-order (1:100,000) has been performed. A series of propagated distance accuracies from a minimally constrained adjustment is now computed.

Line	*S (in)*	*d (in)*	*1:a*
1.2	0.141		17,107
1:121326			
1-3	0.170	20,123	
1:118371			
2.3	0.164	15,505	1:
94.543			

Suppose that the worst distance accuracy is *1:94,543*.

This is not substantially different from the intended accuracy of 1:100,000, which would therefore have precedence for classification. It is not feasible to precisely quantify “substantially different.” Judgment and experience are determining factors.

Now assume that a solution combining survey and network data has been obtained (as per appendix B), and that a variance factor ratio of 1.2 was computed for the survey. This would be reasonably close to unity, and would indicate that the survey checks with the network. The survey would then be classified as first-order using the intended accuracy of 1:100,000.

However, if a variance factor of, say, 1.9 was computed, the survey would not check with the network. Both the survey and network measurements then would have to be scrutinized to find the problem.

Monumentation

Control points should be part of the National Geodetic Horizontal Network only if they possess permanence, horizontal stability with respect to the Earth’s crust, and a horizontal location which can be defined as a point. A 30-centimeter-long wooden stake driven into the ground, for example, would lack both permanence and horizontal stability. A mountain peak is difficult to define as a point.

Typically, corrosion resistant metal disks set in a large concrete mass have the necessary qualities. First-order and second-order, class I, control points should have an underground mark, at least two monumented reference marks at right angles to one another, and at least one monumented azimuth mark no less than 400 m from the control point. Replacement of a temporary mark by a more permanent mark is not acceptable unless the two marks arc connected in timely fashion by survey observations of sufficient accuracy. **Detailed information may be found in C&GS Special Publication 247, “Manual of geodetic triangulation.”**

2.2 Vertical Control Network Standards

When a vertical control point is classified with a particular order and class, NGS certifies that the orthometric elevation at that point bears a relation of specific accuracy to the elevations of all other points in the vertical control network. That relation is expressed as an elevation difference accuracy, b. An elevation difference accuracy is the relative elevation error between a pair of control points that is scaled by the square root of their horizontal separation traced along existing level mutes.

Table 2.2—elevation accuracy standards

Classifications	maximum elevation difference accuracy
First-order, class I	0.5
First-order, class II	0.7
Second-order, class I	1.0
Second order, class II	1.3
Third-order	2.o

An elevation difference accuracy, b, is computed from a minimally constrained, correctly weighted, least squares adjustment by

B = S/Vd

where

d = approximate horizontal distance in kilometers between control point positions traced along existing level routes

S = propagated standard deviation of elevation difference in millimeters between survey control points obtained from the least squares adjustment. Note that the units of b are *(mm)/V* (km).

The elevation difference accuracy pertains to all pairs of points (but in practice is computed for a sample). The worst elevation difference accuracy (largest value) is taken its the provision 1 accuracy. If this is substantially larger or smaller than the intended accuracy, then the provisional accuracy takes precedence.

As a test for systematic errors, the variance factor ratio of the new survey is computed by the Iterated Almost Unbiased Estimator (IAUE) method described in appendix B. This computation combines the new survey measurements with existing network data, which are assumed to be correctly weighted and free of systematic error. If the variance factor ratio is substantially greater than unity, then the survey does not check with the network, and both the survey and the network data will be examined by NGS.

Computer simulations performed by NGS have shown that a variance factor ratio greater than 1.5 typically indicates systematic errors between the survey and the network. Setting a cutoff value higher than this could allow undetected systematic error to propagate into the national network. On the other hand, a higher cutoff value might be considered if the survey has only a small number of connections to the network, because this circumstance would tend to increase the variance factor ratio.

In some situations, a survey has been designed in which different sections provide different orders *at'* control For these multi-order surveys, the computed elevation difference accuracies should be grouped into sets appropriate to the different parts of the survey. Then, the largest value of b in each set is used to classify the control points of that portion, as discussed above. If there are sufficient connections to the network, several variance factor ratios, one for each section of the survey, should be computed.

Vertical Example

Suppose a survey with an intended accuracy of second-order, class II has been performed. A series of propagated elevation difference accuracies from a minimally constrained adjustment is now computed.

	S	*d*	*b*
line	*(mm)*	*(kin)*	*(mm)/ V (kin)*
1-2	1.57 4	1.718	1.20
1-3	1.743	2.321	1.14
2-3	2.6.47	4.039	1.32

Suppose that the worst elevation difference accuracy is 1.32. This is not substantially different from the intended accuracy of 1.3 which would therefore have precedence for classification. It is not feasible to precisely quantify "substantially different." Judgment and experience are determining factors.

Now assume that a solution combining survey and network data has been obtained (as per appendix B), and that a variance factor ratio of 1.2 was computed for the survey. This would be reasonably close to unity *and* would indicate that the survey checks with the network. The survey would then be classified as second-order, class II, using the intended accuracy of 1.3.

However, if a survey variance factor ratio of, say, 1.9 was computed, the survey would not check with the network. Both the survey and network measurements then would have to be scrutinized to Th~d the problem.

Monumentation

Control points should be part of the National Geodetic Vertical Network only if they possess permanence, vertical stability with respect to the Earth's crust, and a vertical location that can be defined as a point. A 30-centimeter-long wooden stake driven into the ground, for example, would lack both permanence and vertical stability. A rooftop lacks stability and is difficult to define as a point. Typically, corrosion resistant metal disks set in large rock outcrope or long metal rods driven deep into the ground have the necessary qualities Replacement of a temporary mark by a more permanent mark is not acceptable unless the two marks are connected in timely fashion by survey observations of sufficient accuracy. **Detailed information may be found in *NOAA Manual NOS NGS* 1, "Geodetic bench marks."**

3. specifications

3.1 Introduction

All measurement systems regardless of their nature have certain common qualities. Because of this, the measurement system specifications follow a prescribed structure as outlined below. These specifications describe the important components and state permissible tolerances used in a general context of accurate surveying methods. The user is cautioned that these specifications are not substitutes for manuals that detail recommended field operations and procedures.

The observations will have spatial or temporal relationships with one another **as given in the "Network Geometry" section.** In addition, this section specifies the frequency of incorporation of old control into the survey. Computer simulations could be performed instead of following the "Network Geometry" and "**Field Procedures" specifications**. However, the user should consult the National Geodetic Survey before undertaking such a departure from the specifications.

The "Instrumentation" section describes the types and characteristic of the instruments used to make observations. An instrument must be able to attain the precision requirements given in **"Field Procedures**.

The section "Calibration Procedures" specifies the nature and frequency of instrument calibration. An instrument must be calibrated whenever it has been damaged or repaired.

The "Field Procedures" section specifies particular rules and limits to be met while following an appropriate method of observation. **For a detailed account of how to perform observations, the user should consult the appropriate manuals.**

Since NGS will perform the computations described under **"Office Procedures**," it is not necessary for the user to do them. However, these computations provide valuable checks on the survey measurements that could indicate the need for some reobservations. **This section** specifies commonly applied corrections to observations, and computations which monitor the precision and accuracy of the survey. It also discusses the correctly weighted, minimally constrained least squares adjustment used to ensure that the survey work is free from blunders and able to achieve the intended accuracy. Results of the least squares adjustment are used in the quality control and accuracy classification procedures. The adjustment performed by NGS will use models of error sources, such as crustal motion, when they are judged to be significant to the level of accuracy of the survey.

APPENDIXES

APPENDIX-A

Traverse stations croca

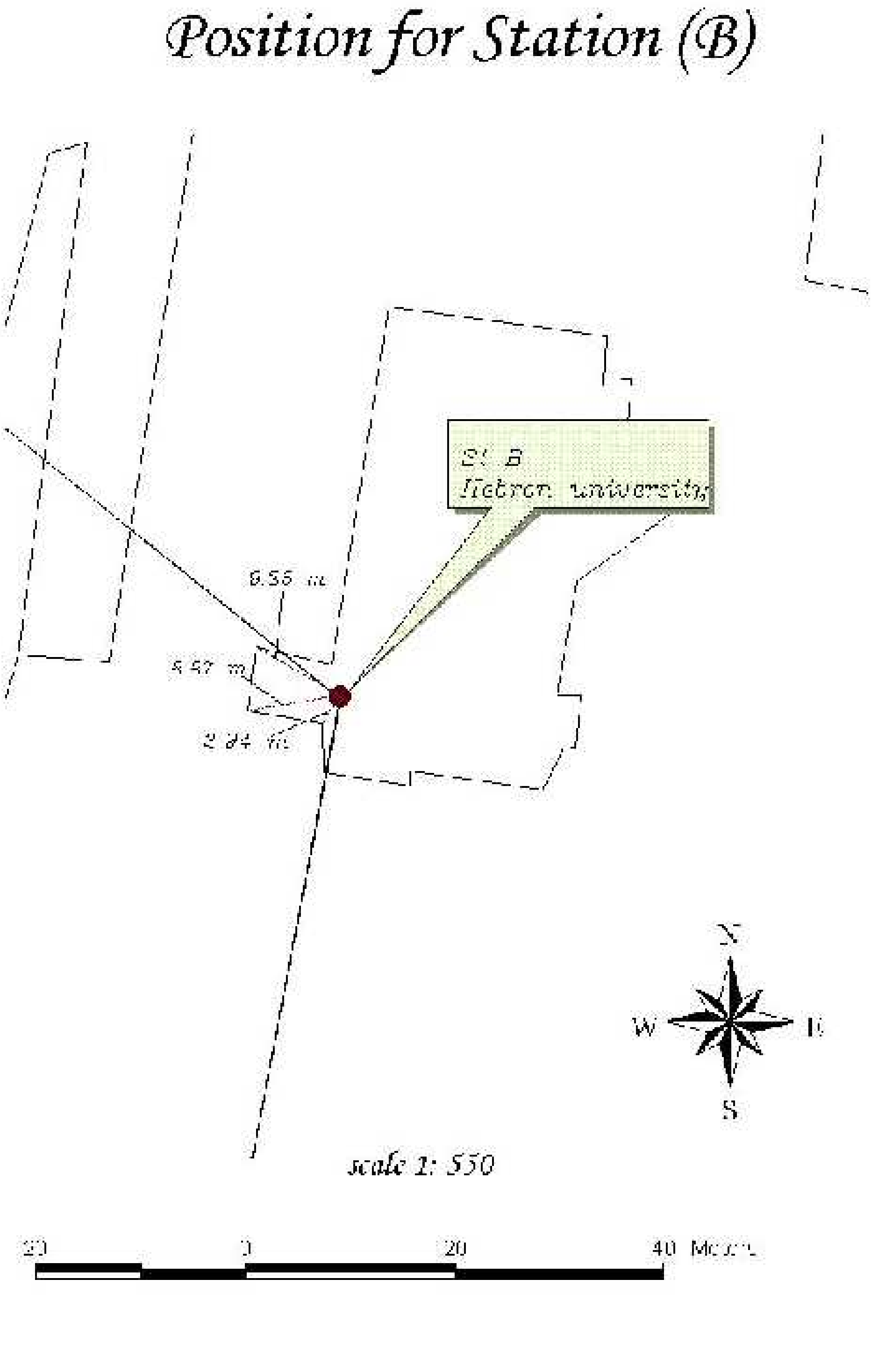


Figure A-1: Kroca for Station B.

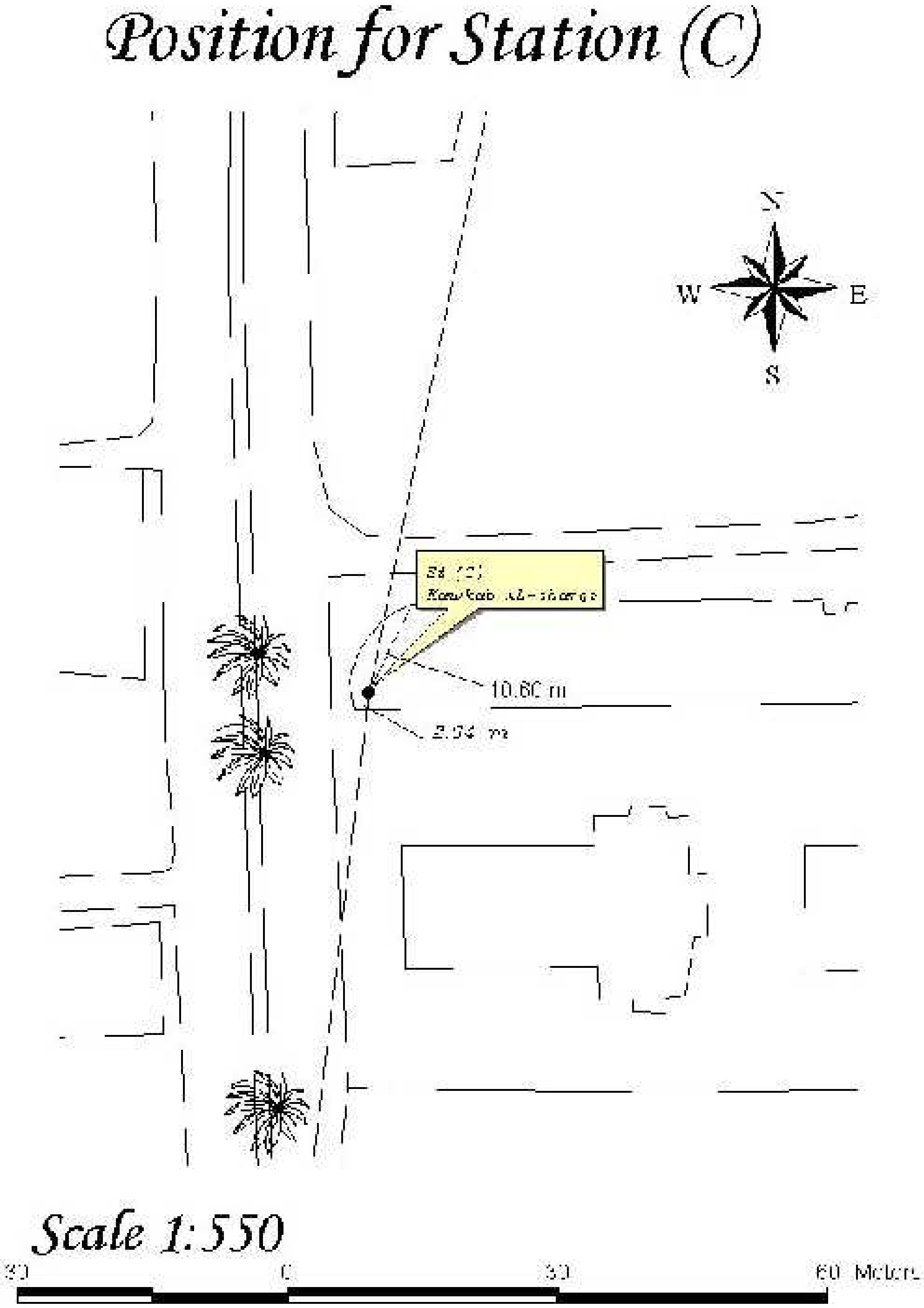


Figure A-2: Kroca for Station C.

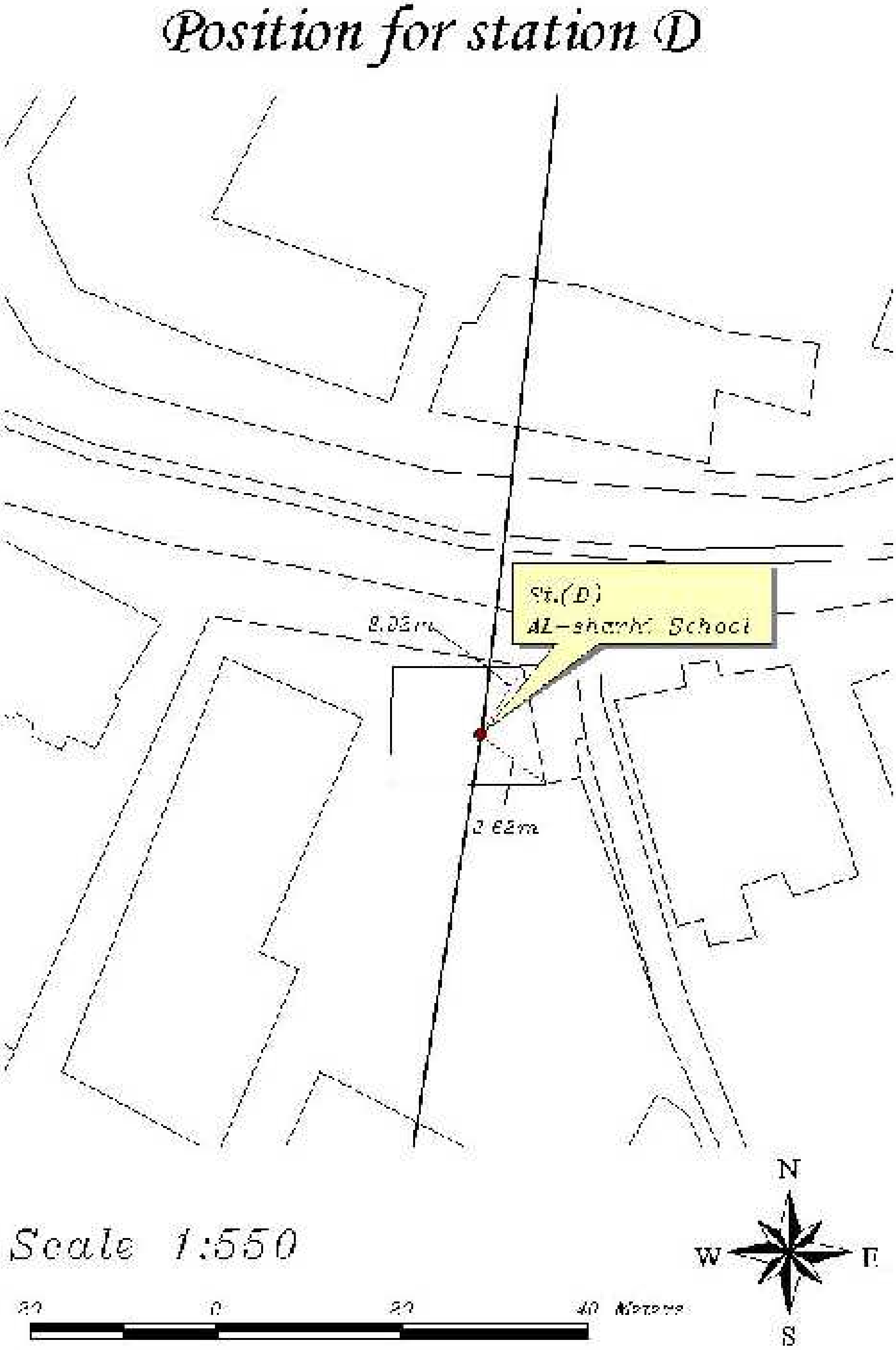


Figure A-3: Kroca for Station D.

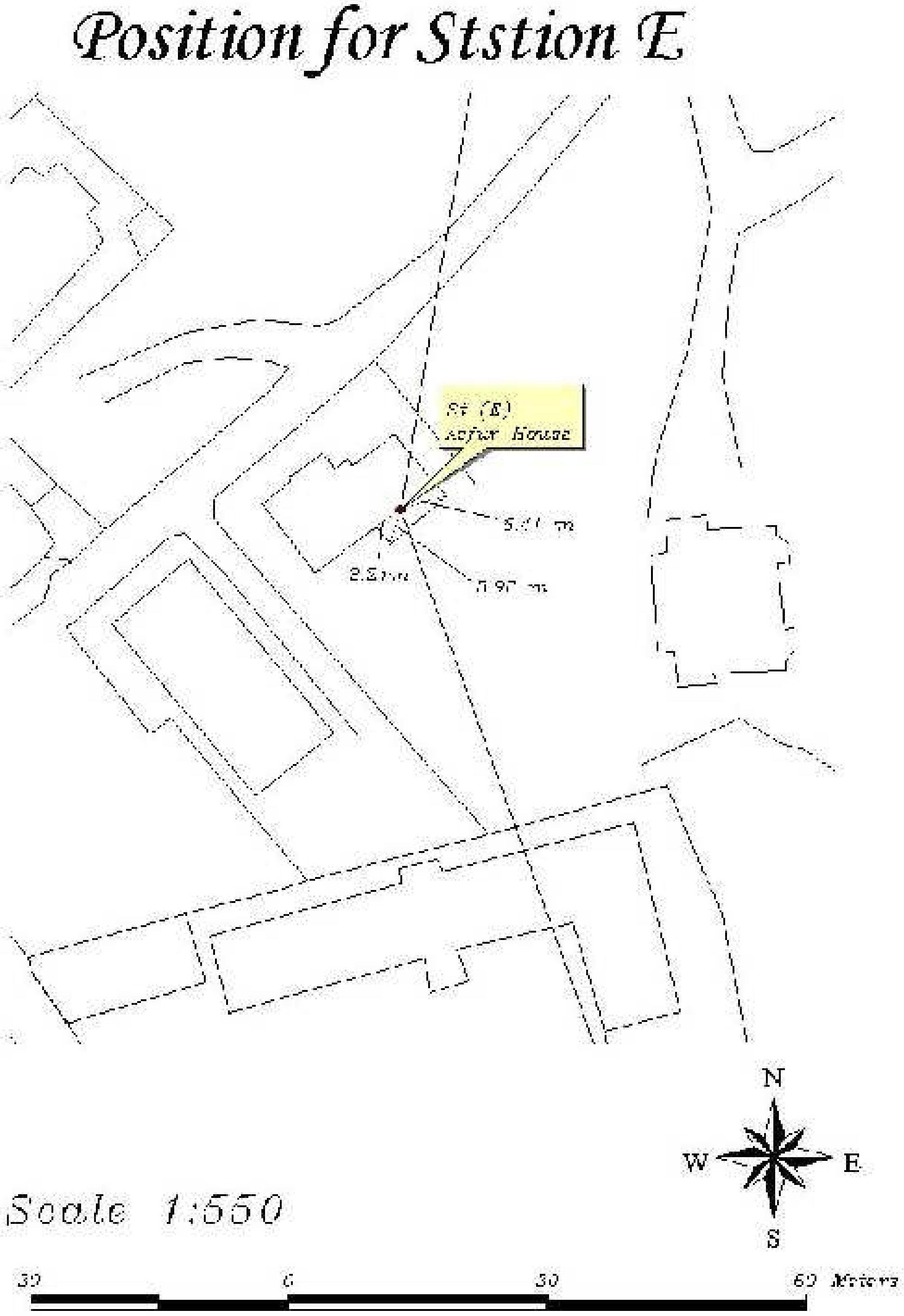


Figure A-4: Kroca for Station E.

APPENDIX-B

J-matrix and table of t-distribution and field survey observation tables

APPENDIX-C

Computer programs used in project

C-1 Arc View 3.2

It is a Geographic Information System "GIS" computer package that produced by the Environmental Systems Research Institute “ESRI”, a leading company in GIS Industry. It is one version of many series versions of GIS packages produced by Esri.

C-2 Soft Desk 8

It is a civil survey program that allows solving all surveying operations from calculations and drawing and field data capture to be entered with it. It must be installed only over AutoCAD 14.

C-3 AutoCAD 2002

This Auto Cartographic Aided Drawing program is a drawing program used for drawing any thing, like: Land representation, Parcels, buildings, and other application like machine drawing in 3D model...etc.

It is same as AutoCAD 14, and 2000, a new version of CAD and the more powerful one to make powerful drawing.

C-5 MathCAD

It is a program for mathematical calculation.

Table B-6: FIELD SURVY OBSERVATION TABLE

						Zenith Angle (B.S) ° ' "		Zenith Angle (F.S) ° ' "		Horizontal Angle ° ' "			
Back sight	**Occupied**	**Fore sight**	**Slope Distance**	**Elevation Difference**	**Horizontal Distance**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Face Right**	**Face Left**	**Instrument Height**	**Reflector Height**
	ST 106												
ST 105			1606.158	47.54	1605.449	88 18 14	271 41 44			312 29 00	312 28 58	1.65	1.65
						88 18 12	271 41 42			312 28 58	312 29 00	1.65	1.65
						88 18 10	271 41 40			312 28 56	312 28 58	1.65	1.65
		ST 107						91 48 28	268 11 30				
								91 48 26	268 11 32				
								91 48 24	268 11 34				
		1	9.963	-3.157	9.450					270 11 58		1.65	
		2	647.598	-5.944	647.571					26 30 42		1.65	
		3	634.986	-5.954	634.958					26 27 20		1.65	
		4	667.703	-23.170	667.366					16 03 36		1.65	
												1.65	

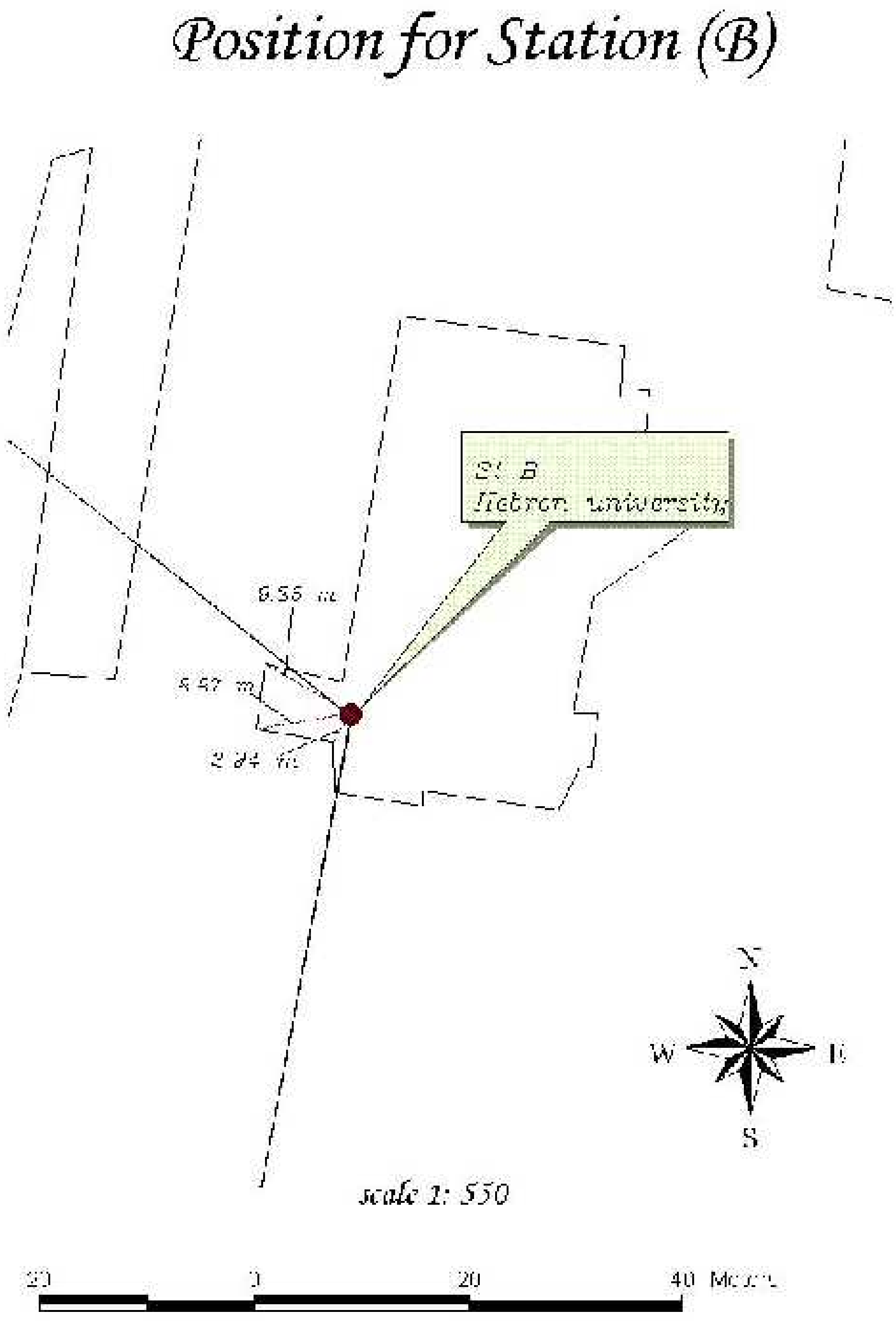


Figure A-1: Kroca for Station B.

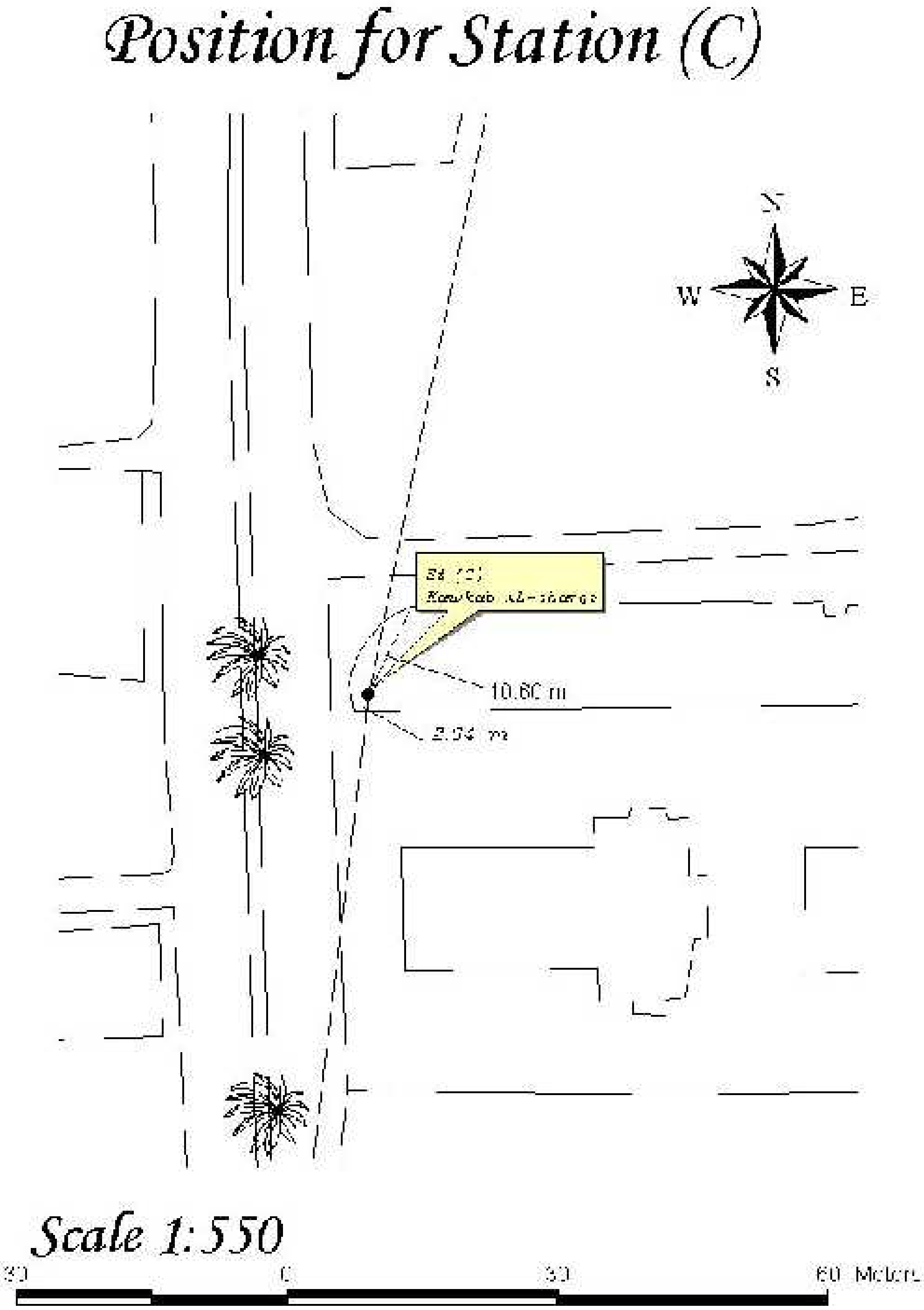


Figure A-2: Kroca for Station C.

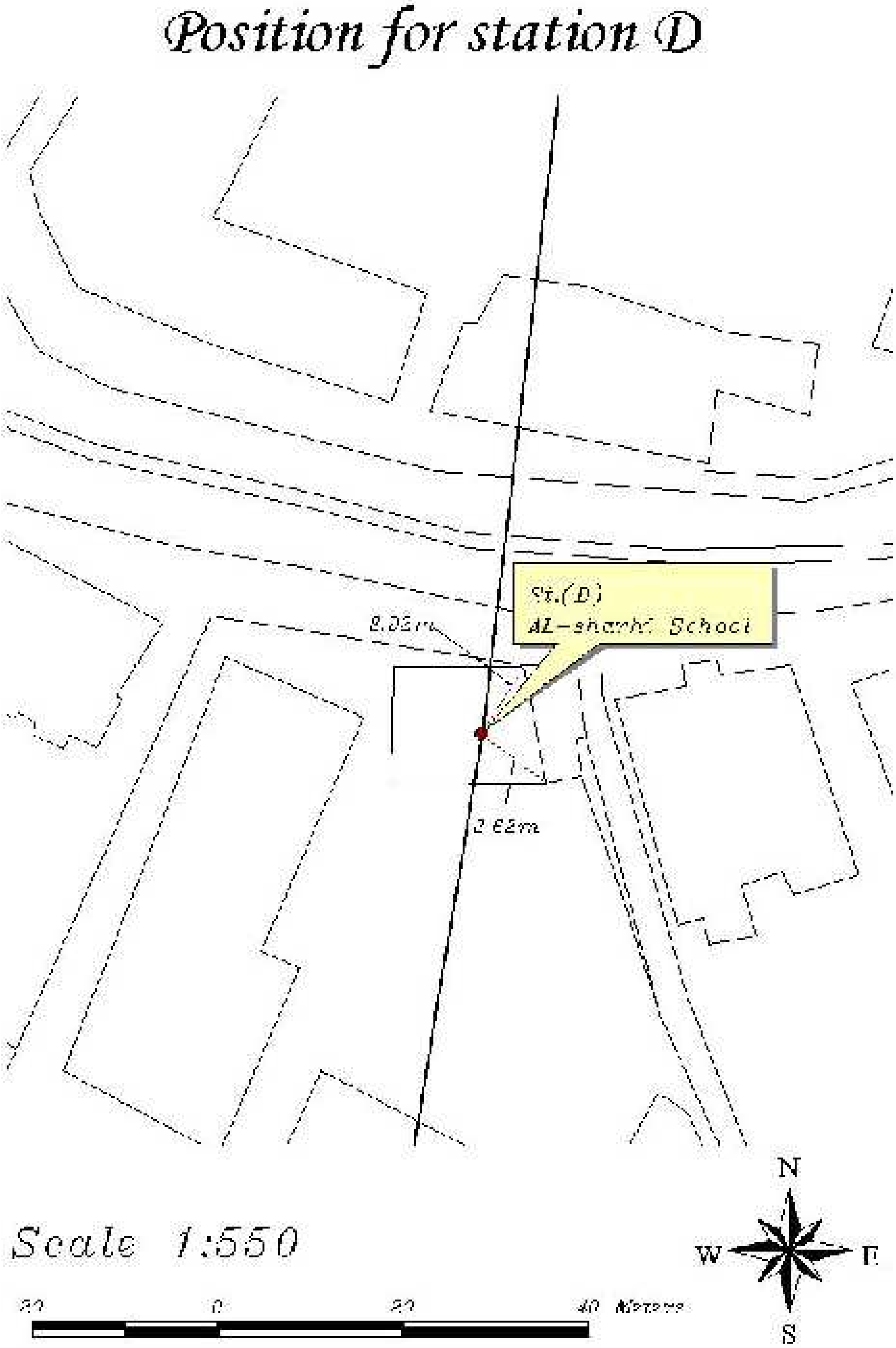


Figure A-3: Kroca for Station D.

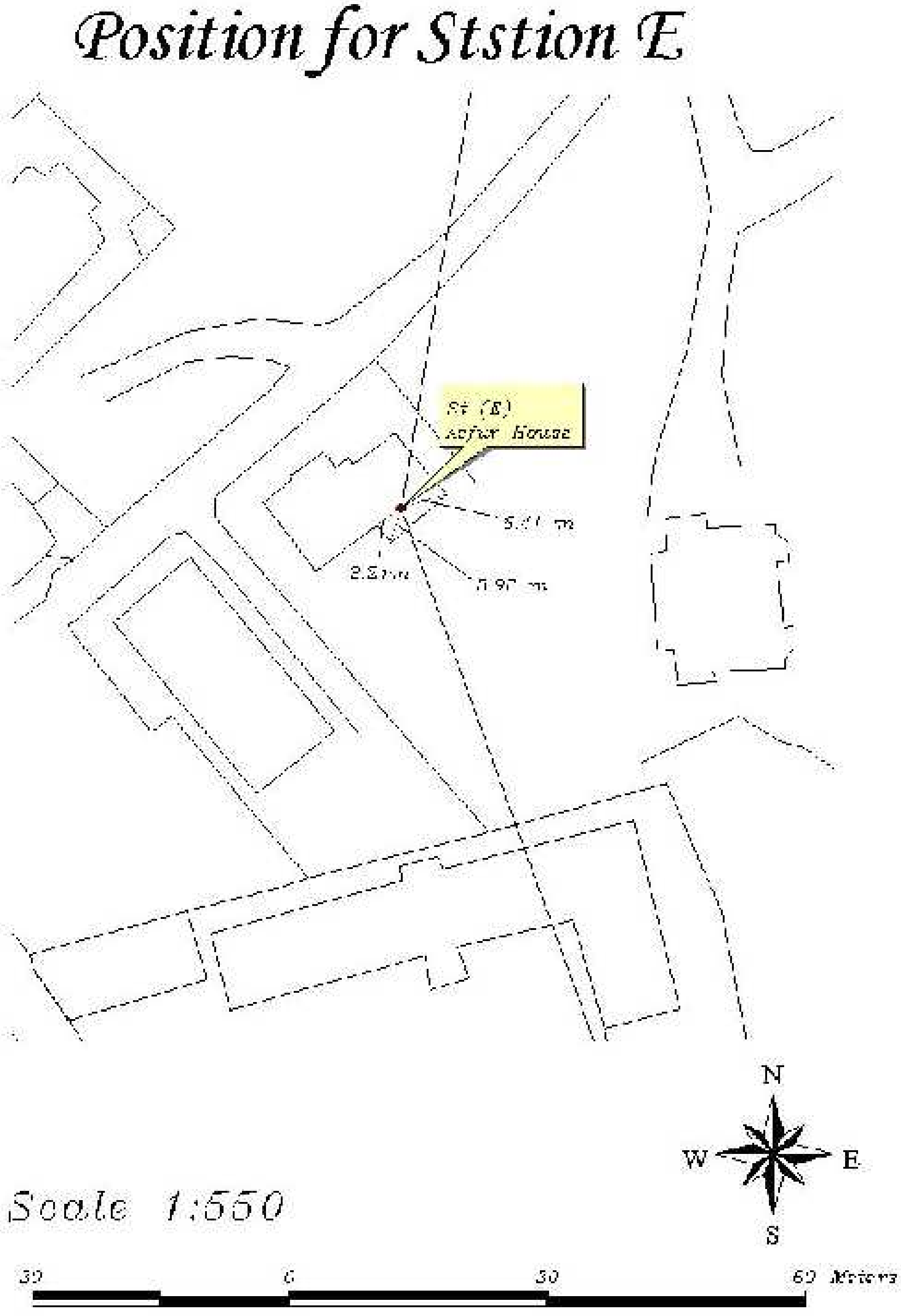


Figure A-4: Kroca for Station E.