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Geographic information system for a small city: Roland, Iowa

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Geographic information system for a small city: Roland, Iowa

by

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CHAPTER 1. INTRODUCTION

The purpose of my thesis project was to develop a Geographical Information System (GIS) for two subdivisions within the town of Roland, Iowa.

The intent of a GIS is to provide information to the user assisting him in making decisions regarding matters linked to the earth's surface. The value of an efficient GIS has sparked interest and application development in all fields from agriculture to social science to engineering and more. The increased accessibility to computing power for data processing via the personal computer and work station level computer has greatly enhanced the development of GIS to meet the multitudes of application opportunities.

A list of essential components that make up a GIS are given in (10). The elements a GIS must contain are: data acquisition, preprocessing, data management, manipulation and analysis, and product generation (10). For the individual responsible for developing a GIS, these elements can serve as groupings of tasks to accomplish. These elements served as a road map for the completion of my project. Specific tasks of what was accomplished within the elements will be discussed in Chapter 2. How these specific tasks were accomplished will be discussed in the remaining chapters.

The software package used was the UltiMap Engineering, Mapping, Planning and Geographic Information System program (version 2.0.57, released May 25, 1990).

The hardware system used was an Apollo workstation, utilizing the AEGIS operating system, located in the ISU Engineering Computation and Information Laboratory (ECIL). Peripheral hardware included a CALCOMP 2500 digitizing tablet and a Hewlett Packard Draftmaster I Plotter.

Chapter 2 of the paper discusses basic concepts of what constitutes a GIS and an overview of the capabilities of the UltiMap GIS software package. Chapter 3 describes the different surfaces used in geodesy as well as geodetic datum. This information will be referred to throughout the rest of the paper. Chapter 4 describes how GPS works and how it was employed for the study. Chapter 5 discusses the concepts of the state plane coordinate systems. Chapter 6 shows how the data provided by GPS were converted to usable data in the state plane coordinate system and briefly describes how the coordinate base map was constructed using the AutoCAD drafting program. Chapter 7 shows how photogrammetry and existing utility plan and profile drawings were used to provide information for the GIS. Chapter 8 describes the use of the UltiMap program to develop the GIS for Roland. Chapter 9 presents conclusions and recommendations.

The project succeeded in its goal to develop a working GIS for Roland. Techniques for the development of a coordinate base map were established. The work also established the important contribution provided by GPS surveying and photogrammetry techniques. The efficiency, data storage/retrieval capability, and analytical capability of an automated GIS were shown. More details of what was accomplished are discussed in the conclusions.

CHAPTER 2. GEOGRAPHIC INFORMATION SYSTEM AND UltiMap

Geographic Information System

There are many definitions of what a Geographic Information System (GIS) actually is. One very broad definition is that a GIS is a facility for preparing, presenting, and interpreting facts that pertain to the surface of the earth (12). This definition allows for manual (analog) as well as an automated (based on a digital computer) system to be called a GIS (10). Whether the system is manual or automated, it begins with some sort of base map which provides the geographical reference to which all other information is oriented. Other spatial information, such as utilities, building outlines, or contours, is then added to this base map in a manner which can be thought of as adding layers of information to the base map (see Fig. 2-1). Nongraphical attribute data pertaining to the graphics are added to the system also.

Following the above description, Roland, and most other towns across the nation, has a manual GIS already in operation. Roland had a city map prepared in 1988 which serves as their base map. They have most of the official subdivision plats and most plan and profile drawings for their streets and utilities which could be thought of as additional layers to the base map. Attribute data regarding such things as parcel ownership or current assessed property value is available by making a phone call to the county assessor in Nevada, Iowa. By referencing the base map, walking to the second floor map cabinet, or making a few phone calls, the city clerk could gather information from the system and make a decision. My goal was to demonstrate, with the aid of computer power, how this existing GIS could be made more efficient.

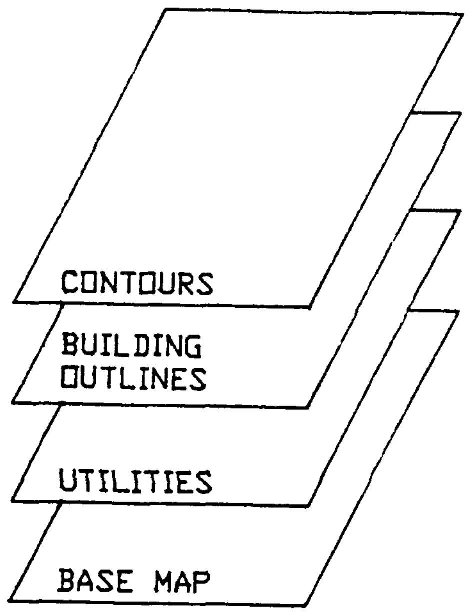

Figure 2-1. Spatial information layering concept

With only the rarest of exceptions, any reference to GIS in the literature or among professionals in the field assumes a computer based system. In light of this, the set of essential functional components of a GIS listed in the introduction can serve as a better definition of what a contemporary GIS is.

The essential elements a GIS must have again are (10):

1. The capability to acquire data. This is the process of identifying and gathering data pertaining to your application.

2. The capability to preprocess data. This is the process of putting all data into a format for entry into the GIS.

3. Data management capability. Processes of creating and providing access to the database. Includes security provisions.

4. Capability to manipulate and analyze data. Processes which create new data from existing data.

5. Capability to generate output products. Processes which produce soft copy (on screen) or hard copy output in a particular format.

The system's configuration of software and hardware is designed to accomplish all but the first element (see Fig. 2-2).

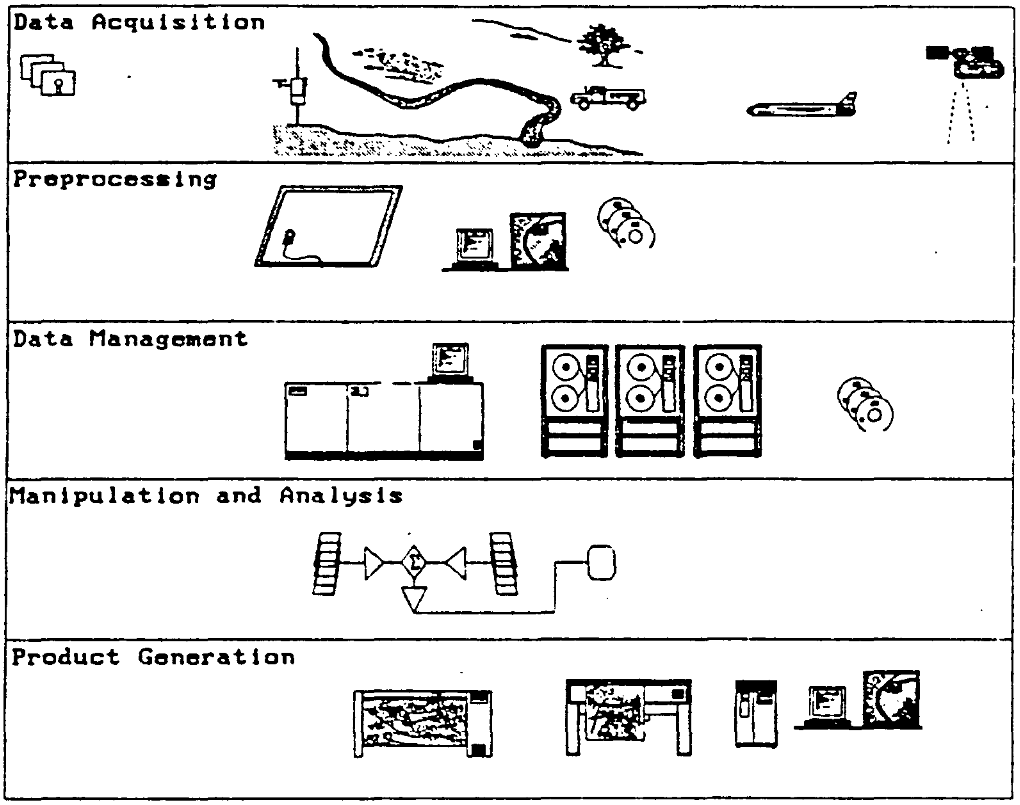

Figure 2-2. The five elements of a GIS (10).

The project was completed by accomplishing tasks within each of these element areas. The GIS was completed primarily in a sequential mode with some concurrent data acquisition and preprocessing. Tasks were organized as follows:

1. Data Acquisition

    a. Determining what data to include in the GIS
    b. GPS surveying
    c. Gathering of subdivision maps and utility plans
    d. Gathering nongraphic attribute data
    e. Obtaining aerial photography

2. Preprocessing

    a. Combining GPS data and subdivision maps into a coordinate base map in the Iowa North Zone State Plane Coordinate System in AutoCAD.
    b. Creation of a photogrammetric stereomodel from aerial photographs
    c. Digitizing features obtained from photogrammetry onto the base map
    d. Digitizing utility information onto the base map
    e. Translating base map files from AutoCAD into UltiMap

3. Data Management

    a. Entering attribute data into UltiMap database

4. Manipulation and Analysis

    a. Using UltiMap's processes to achieve application goals

5. Product Generation

    a. Using UltiMap's processes to provide soft and hard copy output. Hard copy output included attribute listings and map plots.

Many commercial GIS software packages are available with many different capabilities and features. One of the more important features of a GIS software package is what spatial data structure it uses. The type of data structure may affect both the storage volume and processing efficiency of the system. Knowledge of the type of data structure the software is based on will have an impact on what existing databases you might be able to import for use in your GIS. Obviously, the two must be compatible. A brief discussion of spatial data structures follows. More details can be found in (10).

There are two major categories of spatial data structures. These are raster and vector structures. In raster structures, a value for the parameter of interest, land cover type for example, is developed for each picture element (pixel) within the limit of your area of concern (10). A line would be stored as a series of pixels lined up together from the starting point to the ending point. Satellite remote sensing data is processed in a raster structure.

Vector data structures are based on knowing the locations of basic entities such as points, lines, and circles. A line might be stored as a starting and ending point with known X,Y coordinates. Polygons are stored as areas completely enclosed by lines. A circle might be stored as a center point with a radius in a vector structure, versus being stored as a series of pixels that form the perimeter of the circle in a raster structure.

The GIS user must match the spatial data structure of the system with his application needs. Some software can process both types of data structures. In my application for Roland, knowing the specific locations and lengths of features was important. Almost all spatial features of concern were linear or polygons bounded by straight lines. These factors made a vector data structure more favorable. UltiMap uses a vector spatial data structure. The UltiMap program will be discussed in more detail later in the chapter.

The primary goal of the project was to develop a functioning GIS for the city with a secondary goal being to develop techniques in producing a coordinate base map for the GIS using GPS surveying versus simply digitizing existing city and subdivision maps. Many systematic and random errors result from digitizing existing maps. The map you are digitizing may itself be a second or third generation map including errors from previous digitizing or reproduction. The paper the map is printed on may have contracted or expanded causing inaccuracies. Other errors result from the ability of the operator to place the cross hairs of the digitizing puck on the point he wants to locate and from the electronic capabilities of the digitizing hardware. A coordinate base map avoids these errors. A coordinate base map is one where coordinates of key points on the ground, such as parcel corners and points along road centerlines, are determined through a variety of techniques and entered directly into a computer graphics file. These points are then connected with lines or arcs to form the map.

Dakota County, located in the Twin Cities metropolitan area of Minnesota, has recently completed a coordinate base map using GPS surveying of their entire county to the level of computing lot corner coordinates for all subdivisions. Neighboring Ramsey County and the city of Newark, New Jersey have also used GPS surveying to develop coordinate base maps, but used digitizing or scanning techniques to provide detail of lot corners and lot boundaries. All endeavors were considered successful in terms of map accuracy and cost effectiveness (2,6).

Roland, Iowa is located in Story County, about 12 miles northeast of Ames, Iowa. Roland was chosen as the project site for several reasons. With a population of about 1000 people, Roland is large enough to have a city utilities department which provides water, sewer, and street maintenance for the town. (Electricity is provided by Iowa Electric Light and Power Company.) The department could appreciate the possible benefits of a computer based GIS and served as an invaluable source of data. Roland's corporate boundaries lie directly along lines connecting public land survey monuments with only a slight deviation on the eastern boundary. The common corner of sections 14, 15, 22 and

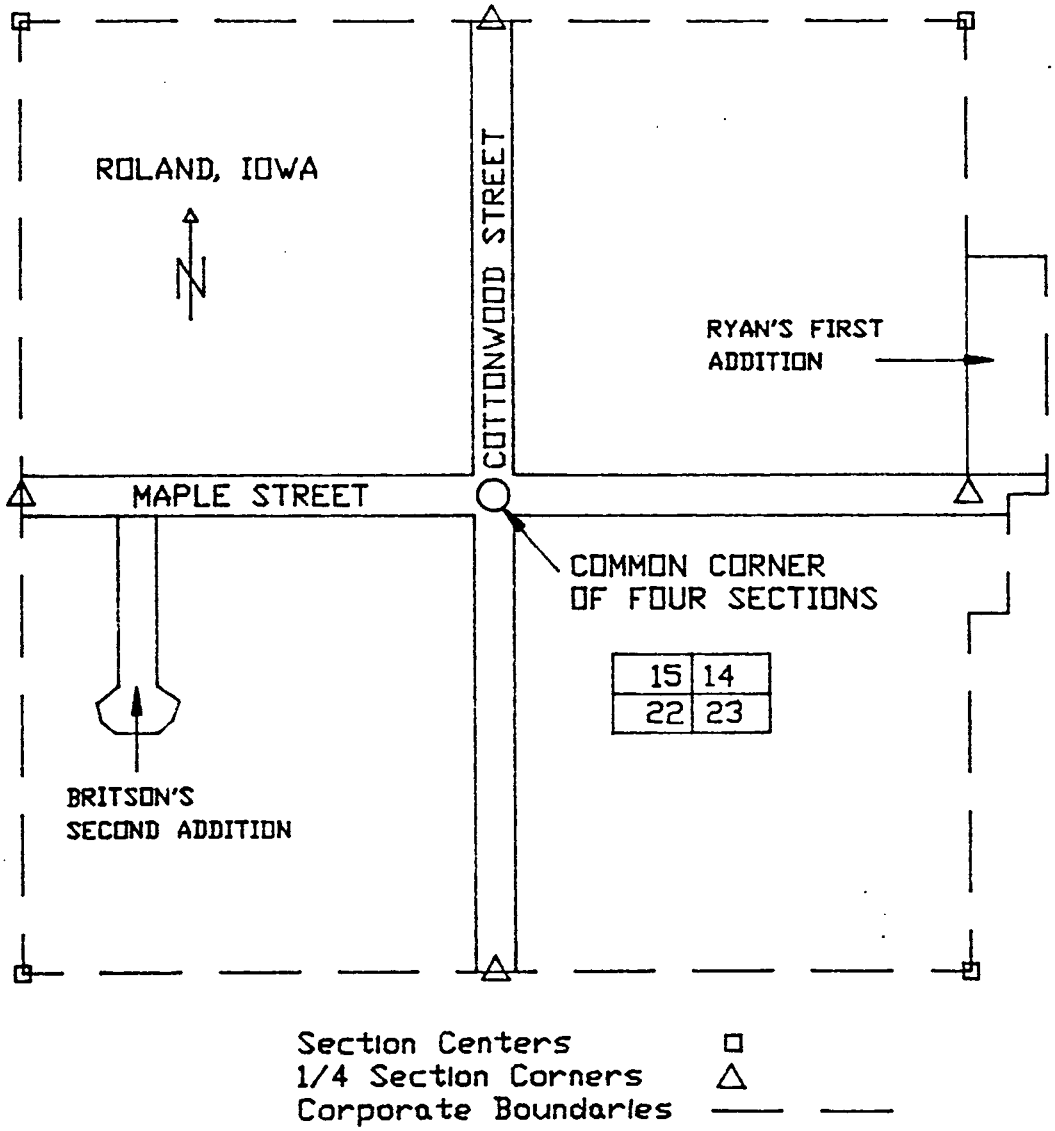

Figure 2-3. Approximate corporate boundaries of Roland, Iowa (not to scale).

23 of Township 85 North, Range 23 West of the fifth principal meridian lies in the middle of town at the intersection of two major roads as shown in Figure 2-3. This greatly helped in the development of the base map. The city's boundaries encompass an area of only slightly more than one square mile which kept the project area within manageable work limits given the time and resources available for the project. Finally, Roland's close proximity to the ISU campus limited travel time and other logistical concerns for the conduct of field surveying work.

Figure 2-3 also shows the two subdivisions that were included in the GIS. The subdivisions are Ryan's First Addition in the northeast part of town and Britson's Second Addition in the southwest part of town.

The UltiMap Program

The UltiMap program meets all software requirements of a GIS to preprocess data, manage data, manipulate and analyze data, and to provide for product generation. The program is suitable for two levels of users. For those with a basic knowledge in computer science, it can be designed to perform data analysis for different GIS applications. The UltiMap user interface also supports those users who wish only to use the results of someone else's design work for a specific application. The user interface is primarily icon driven with some keyboard entry required.

The program consists of eight modules, four of which I used extensively in my project for GIS development. These will be discussed below. The other four modules are application modules such as a Coordinate Geometry (COGO) module and a Road Design System. These modules provide powerful computation capability to produce data for inclusion in GIS or for separate use.

The Interactive Graphics System (IGS) module is the graphics module and is the heart of the system. This module provides a fairly sophisticated level of Computer Assisted Drafting and Design (CADD) capability. Commands within the IGS module are organized under a chapter and page type convention such that every command is entered in an alphanumeric fashion. One major difference between the UltiMap IGS module and a standard CADD program such as AutoCAD is that there are no layers in IGS. Each entity drawn is assigned a specific symbol definition within what is called a symbol dictionary. The visibility of individual symbol definitions is toggled on or off versus layers in AutoCAD. As part of the GIS capability, each entity drawn in IGS is assigned a reference number which can serve as part of a process to tie that particular entity to a record in the database.

Drawing files created in AutoCAD can be imported into an IGS drawing file by utilizing an UltiMAP translator routine. This translation is accomplished using AutoCAD DXF files.

The Revision Control Data Management System (RCDMS) is the database management module of UltiMap. This module allows the building of a database through IGS and other modules. RCDMS allows for many people to be working on a project and for the sharing and merging of their respective data. It also provides for project security through user access options. This module also has a time stamping feature which allows for a historical record of a project.

The Topological Access Generator (TAG) module allows for the creation of schemes to relate graphical features to nongraphical attribute data files. The relationship between graphical and nongraphical data is accomplished by linking reference numbers assigned to an entity in IGS with a particular value in an index field of an attribute data file. Figure 2-4 shows the tagging concept.

Reference Numbers		Index Field	Line Length	Lot No.
100.10	<————>	LIN01	100	14
100.20	<————>	LIN02	110	15
100.30	<————>	LIN03	150	16
			Data File	

Figure 2-4. Tag scheme relating reference numbers to values in an index field.

The information to the right of the arrows represents a small data file with the first field being defined as the index field. The remaining fields in the data file represent attribute data pertaining to graphical entities, perhaps lot lines in a subdivision. The reference numbers on the left side of the arrows are the unique reference numbers assigned to three different graphical entities, lot lines in our example.

The user specifies how the values in the index field will be linked to a reference number. In Figure 2-4, reference numbers 100.10, 100.20, and 100.30 are linked with index field values LIN01, LIN02, and LIN03, respectively. When a particular index field value is indicated through different UltiMap processes, all attribute data for the record with that index field value become available for processing.

This linking, or tagging, concept to tie graphical and nongraphical data is a basic concept in all GIS software. Different programs use different procedures.

The Q-Star Fourth Generation Language (Q-star) module is the primary module used in the development of GIS. It is within this module new or existing data files are prepared for processing and the system receives its instructions on what processing is to be done to the available data. Q-star consists of three major submodules including Q-link, Q-join, and Q-view.

Q-link is the submodule used to define the physical layout of a nongraphic datafile. New or imported existing files, such as a DIME file, must be defined. This definition tells the system what data are available for processing. Q-join is the module used to manipulate information files for efficient data extraction and file updating. Q-view is the most important submodule of Q-star and is the most important submodule in the creation of GIS.

Within Q-view, the user programs the system as to what data will be processed and what the processing will be. Q-view allows not only for retrieval and display of data, but also analysis of existing data to create new data for management purposes. Q-view allows for combined analysis of graphical and nongraphical data or nongraphical data analysis by itself using spreadsheet type techniques. Within Q-view, the user also specifies which device the output of a particular process will be sent to and the output format.

The UltiMap program is sold by the ULTIMAP Corporation of Minneapolis, Minnesota. Hennepin County of the Metropolitan Twin Cities area spent several years actually developing the program, then sold the program to the ULTIMAP Corporation in the mid-1980's. ULTIMAP provided the software to Iowa State University (ISU) along with the documentation at no charge and provided software and documentation updates as they were developed. I attended three days of formal training on the use of the program at the ULTIMAP headquarters in Minneapolis. Support throughout the project was obtained free of charge on the consumer help line. This arrangement was adequate for project completion.

CHAPTER 3. GEODETIC SURFACES AND DATUM

Geodetic Surfaces

Geodesists routinely contend with three surfaces in their work. These three surfaces are shown in Figure 3-1 and are the earth's surface or topographic surface, the geoid, and the ellipsoid.

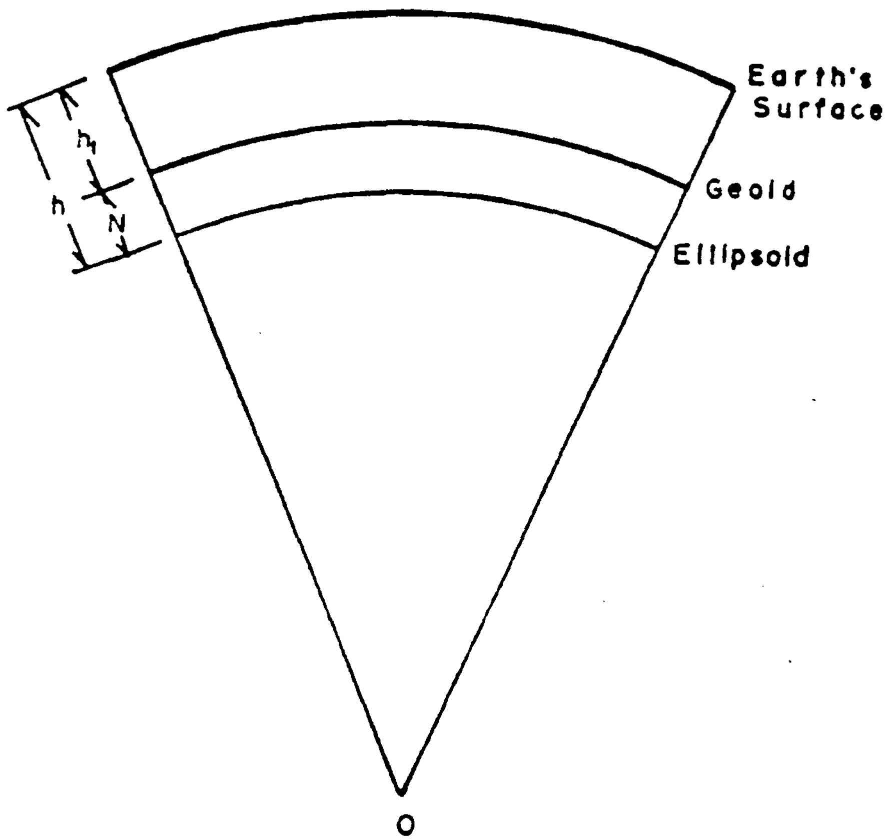

Figure 3-1. Three surfaces dealt with in geodetic calculations (5).

The geoid is the inland projection of the surface of the sea and is normal to the direction of gravity at all locations. Because of differences of mass within the earth, the geoid warps and undulates forming an irregular surface. Because of these undulations, the geoid may be above or below the ellipsoid at different locations. The ellipsoid is a surface which closely fits the undulating geoid surface but yet forms a uniform geometric figure which allows for relatively straightforward mathematical computations. The ellipsoid is formed by rotating an ellipse about its minor axis. The ellipsoid may also be referred to as the "ellipsoid of revolution" or the "spheroid" (5). In Figure 3-1, N is the geoidal height or geoidal undulation. N is positive if the geoid is above the ellipsoid and negative if the geoid is below the ellipsoid. h is the distance of the topographic surface above the ellipsoid or the ellipsoidal height. h_1 is the height of the topographic surface above the geoid which can be equated to the height of the topographic surface above mean sea level.

A fourth surface is introduced when one wishes to create a flat portrayal of the features of the earth's curved surface. This is a map projection surface. Chapter 5 is devoted to the discussion of the Lambert conformal projection which was used in this project.

Several different ellipsoids have been developed in an attempt to obtain a good fit to the geoid in different regions of the world. They have slightly different shapes based on their different values of the parameters which describe an ellipse, a and f. a is the semimajor axis of an ellipse and f is a measure of the flattening of an ellipse where:

$$f = 1 - \frac{b}{a}$$

a is the semimajor axis of the ellipse

b is the semiminor axis of the ellipse

Geodetic Datum

Latitude, longitude, and ellipsoidal elevations are the geodetic or "geographical" coordinates of a point and are based upon a geodetic datum. A geodetic datum consists of a particular reference ellipsoid tied in some manner to the physical earth (5). There are several geodetic data in use and the geodetic coordinates of a point will differ slightly from datum to datum. The datum I used for my study was the North American Datum, 1983, or NAD 83.

CHAPTER 4. GLOBAL POSITIONING SYSTEM

Satellite surveying using the Global Positioning System (GPS) was utilized in this study to establish ground control for base map and photogrammetry needs. Figure 4-1 shows the 12 control points required for the project.

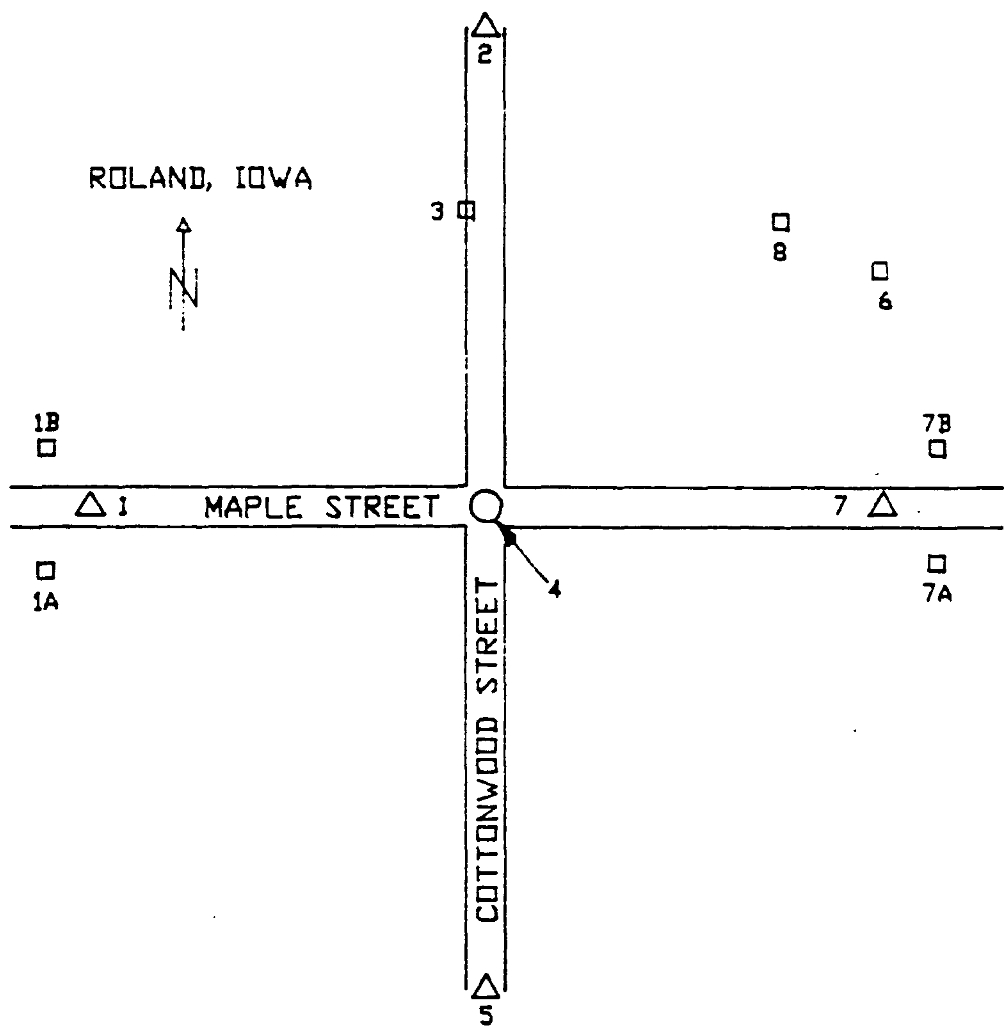

Figure 4-1. Roland control points

All points except 1 and 7 were physically occupied with a signal receiving antenna. Heavy traffic precluded occupation of these points with GPS equipment. Their locations were determined using intersection techniques described in Chapter 6. A discussion of GPS follows.

Background

The Global Positioning System is a Department of Defense (DOD) satellite navigation system. Development began in the mid-1970s and all satellites of the planned constellation should be in place by 1993. The target constellation will have 24 satellites, 21 operational and 3 spares (11). Currently, there are 16 operational satellites. With the current situation, the signals from three satellites can be received at any one point on earth for about 22 hours every day. The signals from four satellites can be simultaneously received for about 15 hours every day. Since precise 3-dimensional positioning requires four satellites, this presents somewhat of an inconvenience. When the full constellation is in place, four satellites will be visible from any point on the earth 24 hours a day (14).

The satellites orbit 20,000 kilometers above the earth and circle the earth twice daily.

How GPS Works

GPS operates on the basic concept of resection to determine positions. Knowing the X,Y,Z coordinates of at least three satellites and the distance from each satellite to an unknown point, the X,Y,Z coordinates of the unknown point can be solved. A fourth satellite is usually observed to solve for a clock error term which will be discussed later.

Distances are determined using electronic signals emitted from the satellites. All GPS satellites transmit two carrier signals at frequencies L1 = 1575.42 MHz and L2 = 1227.6 MHz. These codes are modulated with two other signals. The L1 band is modulated with a C/A code or Standard Positioning Service (SPS) and a P code or Precise

Positioning Service (PPS). The L2 band is modulated only with the P code. The P code is only available to military users (9). The carrier signal also provides information regarding the health of the satellite and parameters used to compute the location of the satellite. Figure 4-2 shows the two bands with the modulations.

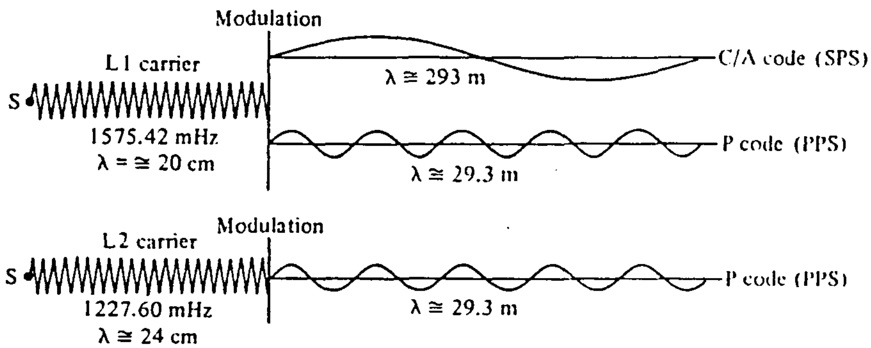

Figure 4-2. Signals broadcast by GPS satellite (9)

These signals are captured by an antenna and passed through a cable to an electronic box about the size of a large shoebox. This box is called a receiver and processes the signal data immediately or stores it for post processing after the field observation. The antennas and receivers used in this study were manufactured by the ASHTEC Corporation. Figure 4-3 shows a typical set-up with the antenna in the background and the operator looking at the receiver.

Distances, or ranges, from the satellite to the point in question are determined by one of two techniques or a combination of these techniques. These are pseudoranging and carrier phase ranging (14).

Figure 4-3. Typical GPS antenna and receiver set-up

In pseudoranging, the time is measured for a particular coded portion of the C/A signal to travel from the satellite to the antenna. The range to each observed satellite is then found from:

$$r = c(\Delta t)$$

where r is the pseudorange, c is the velocity of the propagation of the signal (186,000 mi/sec), and Δt is the measured time from transmission to reception. A distance equation of the form below can then be written for each of the three satellites:

$$r_i = [(X_{si} - X_A)^2 + (Y_{si} - Y_A)^2 + (Z_{si} - Z_A)^2]^{1/2}$$

r_i = the computed pseudorange between the receiver and the ith satellite

X_{si}, Y_{si}, Z_{si} = known coordinates of the ith satellite

X_A, Y_A, Z_A = sought after unknown coordinates of the ground point

This technique requires an absolute synchronization between a clock in the satellite and a clock in the receiver. Because this is impossible, a small clock error exists in the computed range. This clock error can be solved for by observing a fourth satellite which allows a fourth distance equation to be written (see Figure 4-4). Thus, the four unknowns can be solved with the four equations. Each distance equation would then have the form:

$$r_i + \Delta r_i = [(X_{si} - X_A)^2 + (Y_{si} - Y_A)^2 + (Z_{si} - Z_A)^2]^{1/2}$$

The new term, Δr_i, being the clock error term between the ground point and the ith satellite.

Carrier phase ranging provides more accurate range measurements by utilizing the carrier wave for measurement. Carrier phase ranging was used for this study. Advantage is made of the fact that any moment in time, the distance between the satellite and receiver actually consists of an integer member of complete carrier wave phase cycles plus some fraction of a phase cycle. The fraction of phase can be measured to 1% ($\cong$ 2 mm for the L1 band) by the receiver (14). Determining the integer number of phase cycles is the drawback to carrier phase ranging and is called the cycle ambiguity problem. The integer number of phase cycles is determined from approximate satellite and receiver locations for the first epoch of the observation. Using the initial integer count plus the receiver ability to

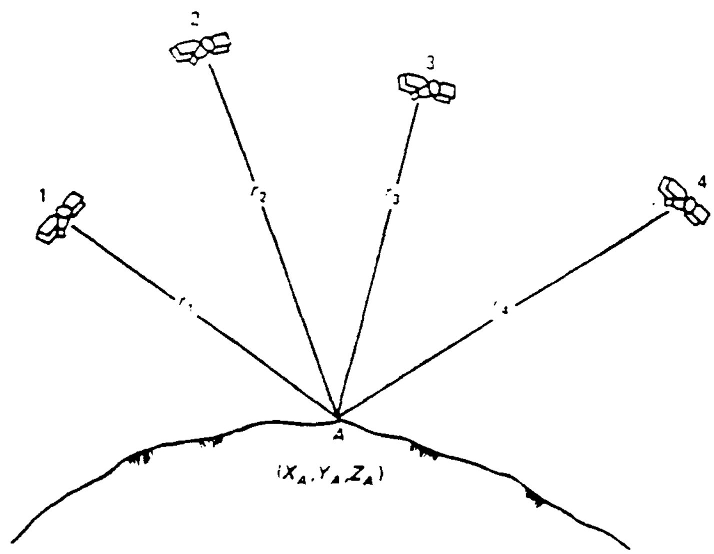

Figure 4-4. Four simultaneously visible GPS satellites (16)

maintain an accumulating sum of complete phases received, the integer number of phase cycles between the satellite and receiver can be determined for subsequent epochs.

Discussion so far has centered around finding the location of a single point using a single receiver. Higher accuracy is obtained by using two or more receivers simultaneously ranging to the same satellites. This is called relative positioning. Differencing the distance equations between two points and the same one or more satellites effectively eliminates clock error and other errors which are assumed to be equal at the two points. This method provides very accurate locations for both points occupied and a very strong relative difference in position between the two points. A method used in this study was to occupy one point whose coordinates were known very well and simultaneously occupy the sought after position with the other antenna. The computed relative difference in position was then added to the coordinates of the known position to determine the coordinates of the sought after point.

Throughout the study, signal data were collected in the field and stored in the receivers for post processing. Post processing involved the downloading of the data from the receiver into a PC and running the data through a series of programs to obtain final results. The output obtained includes the latitude, longitude, and ellipsoidal height of the points occupied, a spatial distance between the two points, X,Y,Z components of the vector between the two points, a geodetic azimuth between the two points, and information regarding the quality of the observation.

An important procedure the post processing accomplishes is to determine the coordinates of the satellites in an earth-centered cartesian coordinate system. The initial coordinates of the satellite are determined in a coordinate system based in the orbital plane of the satellite. These coordinates are then transformed into a celestial coordinate system, and then further transformed into a earth-centered cartesian coordinate system. The position of the point in question is then computed in this coordinate system. These cartesian coordinates are then transformed into spherical, or geodetic, coordinates (latitude, longitude, and ellipsoidal height) of the same system which are provided as final output. These geodetic coordinates can then be transformed into whatever local coordinate system the user desires.

GPS provides ellipsoidal height, or the height of the point above the reference ellipsoid. Usually, we are interested in the geoidal height, or height of the point above the geoid (mean sea level). This conversion is made by applying the geoidal undulation, or separation between the geoid and ellipsoid at that particular point (refer to Figure 3-1). Finding the value of this separation, or geoidal undulation, is a fairly tedious computation which most GPS manufacturers have developed a program to accomplish. I used a program provided by the Trimble Navigation Company to determine the geoidal undulation.

Modes of Operation

There are three modes of operation for GPS surveying. These are the static, pseudokinematic, and kinematic modes. All modes were used in this study utilizing two antennas at all times to provide for relative positioning.

In the static mode, the two points are occupied for 90-120 minutes with data being collected continuously at 10 or 20 second intervals. This procedure provides the highest accuracy. Occasional interruption of the satellite signals can be tolerated in this procedure. Typical accuracy for the static mode using pseudoranging is 5-8 meters. For carrier phase measuring, accuracies in the millimeter range should be attainable (14).

The specific techniques in pseudokinematic and pure kinematic modes differ somewhat, but they essentially call for the occupation of one known point with the other receiver "roving" to several unknown points. The pseudokinematic techniques requires each unknown point to be occupied for two short time periods (5 minutes) separated by at least one hour. This technique is supposed to provide similar accuracy to the static technique (1).

The kinematic mode calls for the rover antenna to initially occupy a known point for about five minutes, then moved to occupy the unknown points for only three minutes each. The kinematic technique establishes points to centimeter accuracy.

The kinematic mode requires continuous lock on the signals from at least four satellites. This is required while occupying a point and while in transit to the next point to occupy. Because of this, the route between points must be clear of overhead obstructions such as trees or buildings which may block the signal. The many trees lining the roads in Roland precluded the use of the kinematic mode in all but one situation. Figure 4-5 shows a view looking south on Cottonwood Street which illustrates the lush overhead cover. In the foreground, the antenna is set up over point 4 and point 5 is 1/4 mile down Cottonwood Street in the direction of view. The overhead cover shown here was typical for routes between all control points.

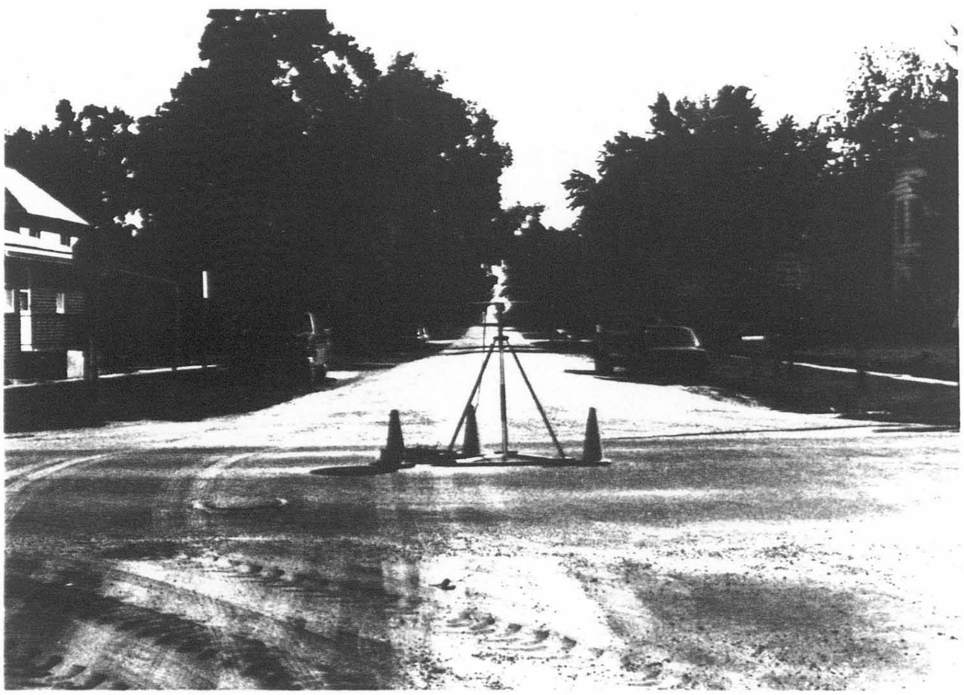

Figure 4-5. Overhead cover looking south on Cottonwood Street

Field Procedures

Static mode observations of approximately 90 minute durations were used to establish the positions of 1B and 7B (see Figure 4-1). Each point was observed relative to two well-known control points on the rooftops of the Town Engineering Building and the Iowa Department of Transportation (DOT) Headquarters. The two slightly different values obtained for the position of each point were adjusted by least squares to obtain final values.

Kinematic procedures were used to help establish the position of 1A. Pseudokinematic procedures were used to establish the remaining points in two separate surveys.

Because the static mode has proven to provide the most accurate results and to provide a link between the different surveys conducted, all kinematic and pseudokinematic surveys were tied to points 1B or 7B. This tie was made by reoccupying either 1B or 7B during a subsequent survey and computing the difference in the coordinate values obtained. This difference, or shift, was applied to all points occupied during the subsequent survey. In general, let λ_s, ϕ_s, h_s be the coordinate values of a point obtained from the static mode and λ_p, ϕ_p, h_p be the coordinate values for the same point obtained through a subsequent pseudokinematic survey. The shift values for each coordinate are computed from:

$$\Delta\lambda = \lambda_s - \lambda_p$$

$$\Delta\phi = \phi_s - \phi_p$$

$$\Delta h = h_s - h_p$$

$\Delta\lambda$, $\Delta\phi$, Δh are then applied to all points located in this subsequent survey. This ties the surveys together.

The standard deviation for each component of a three dimensional position is provided in the post processing output. From these parameters, three dimensional error ellipsoids or two-dimensional error ellipses can be determined for different probability levels. The 95% probability two-dimensional error ellipse parameters for GPS established points are in Table 4-1.

Table 4-1. Error ellipse parameters for GPS located points at 95% probability level

Point	Semimajor axis (cm)	Semiminor axis (cm)	θ
1A	.600	.308	42°
1B	.073	0.29	15.8°
2	.584	.07	-9.22°
3	6.193	2.589	0°
4	.948	.049	0°
5	.697	.07	41.08°
6	6.746	2.383	-15.14°
7A	.027	.095	0°
7B	.083	.022	0°
8	5.494	1.878	-7.58°

The larger ellipses for points 3, 6 and 8 resulted from these points only being occupied once during a pseudokinematic process. One receiver broke down half way through the procedure and was not repairable to redo the observation. The result was the same effect as if each point had only been occupied for a five minute static technique observation.

With the exception of points 3, 6 and 8, all GPS established points achieved first order geodetic control standards in terms of the relative accuracy of the distance between the known point of the surveys, Town Engineering, and the newly established point.

This accuracy was based on the standard error of the distance determined from the roof of Town Engineering to each point. The standard error limits are:

$$.7mm\ \sqrt{K}\ \text{.... } \textit{First Order}$$

$$1.3mm\ \sqrt{K}\ \text{.... } \textit{Second Order}$$

(K is the distance between points in kilometers.)

The standard error of the distance was derived from the positional standard errors for each point; σ_x, σ_y, and σ_z. Using a 2-dimensional example, the relationship between the relative distance standard error and the positional standard errors is derived from error propagation techniques.

Let D = the distance between two points

X,Y = 2-dimensional components of this distance

σ_d = standard error of the distance

σ_x,σ_y = positional standard errors of the established points

Then:

$D^2 = X^2 + Y^2$ (Distance as a function of the X and Y components)

$$2D(dD)=2X(dX)+2Y(dY)$$

$$D(dD)=X(dX)+Y(dY)$$

$$D^2(dD)^2=X^2(dX)^2+Y^2(dY)^2+\ldots$$

$$D^2\sigma_d^2=X^2\sigma_x^2+Y^2\sigma_y^2$$

Assume:

$$\frac{X^2}{D^2}=\frac{Y^2}{D^2}=1 \; and \; \sigma_x^2=\sigma_y^2$$

$$\sigma_d^2=2\sigma_x^2$$

$$\sqrt{\frac{\sigma_d^2}{2}}=\sigma_x$$

From this final relationship, one can determine what positional accuracy must be achieved to meet a required relative distance accuracy.

As an example, in a typical subdivision survey, relative accuracies of ± .01' (± 3 mm) are desired. Letting σ_d = 3 mm and using the relationship derived above, the required positional standard error is:

$$\sigma_x=\sqrt{\frac{9mm}{2}}$$

$$\sigma_x=2.12mm$$

This level of positional accuracy was achieved during the study at both points established using the static mode, but only one point established with the pseudokinematic technique.

From the use of GPS in this project and the example above, it shows that GPS has value in establishing first order horizontal control over long distances (15-30 km), but would not be practical to replace traditional surveying techniques in subdivision surveys.

Assume the need to establish a first order horizontal control point 20 km from a known point. Twenty km is the approximate distance from the roof of Town Engineering to Roland. With static GPS surveying, this could be done in about two hours. A traditional survey crew might be able to establish the new point in about 12 hours. On the other hand, to achieve the .01' relative accuracy in a subdivision survey, it would also take about 2 hours with GPS surveying to establish each new point. With typical traditional surveying equipment, each new point can be established within the acceptable accuracy in about 5 minutes. Thus, GPS becomes impractical.

CHAPTER 5: STATE PLANE COORDINATE SYSTEM

The base map for the GIS was developed in the Iowa North Zone State Plane Coordinate System. A discussion of state coordinate systems follows.

State plane coordinate systems were developed to allow plane surveying techniques to be used over large areas in any direction (9). The state coordinate systems are established with a mathematical relationship to the reference ellipsoid used for geodetic surveys. A fairly dense set of geodetic control points have been established throughout the United States so a local surveyor working in a state coordinate system can tie his surveys to these control points to check his work. Another important point of the state coordinate system is it provides for a single rectangular coordinate system to cover a large area.

Two projections are used in the United States for development of state coordinate systems. These projections are the transverse Mercator projection and the Lambert conformal projection. Since these projections are both plane projections of features from the curved surface of the earth, distortions will occur on both. The distortions in the transverse Mercator projection occurs in the east-west direction and, consequently this projection is used by states that have a short east-west dimension. The Lambert conformal projection's distortions occur in the north-south direction and is used by states with a short north-south dimension relative to their east-west dimension. Iowa uses the Lambert conformal projection. Figure 5-1 shows the Lambert conformal projection.

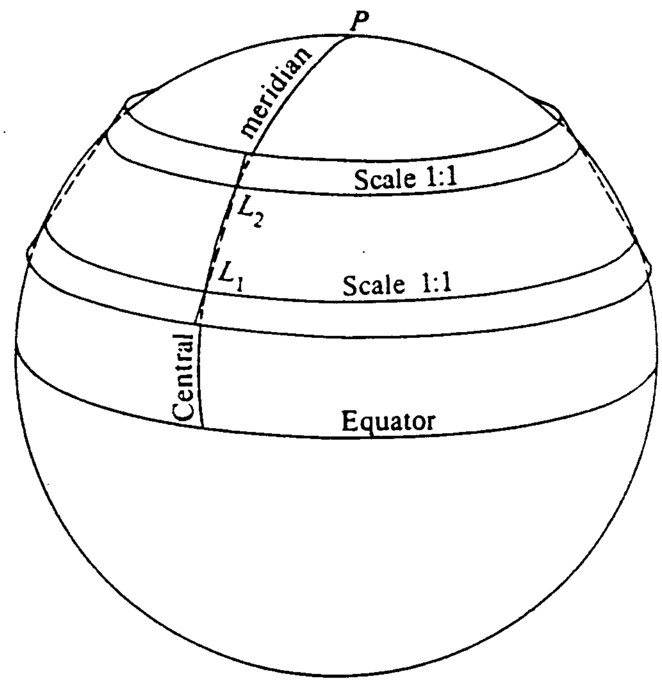

Figure 5-1. Lambert Conformal Projection (9)

Lambert Conformal Projection

The projection is a single conical surface whose axis coincides with the axis of rotation of the earth. The cone intersects the earth's sea level surface at two parallels of latitude that are roughly equidistant from the parallel of latitude at the center of the area of be projected (9). These are called standard parallels. Along these parallels, distances on the projection are the same as their corresponding distances on the sea level surface. Between them, distances on the projection are shorter than corresponding distances at sea level. Outside the standard parallel, distances on the projection are larger than their corresponding distances on the sea level surface. The methods of reducing distances from the topographic surface to sea level surface to projection surface will be discussed later. To limit the difference between corresponding distances at sea level and on the projection to no more than 1 part in 10,000, the projection area of any one zone is limited to 158 miles in the north-south direction (9).

Grid and Geodedic Azimuth

Figure 5-1 shows a feature called the central meridian. The central meridian is located near the center of the projected area.

The central median and all lines parallel to it on the projection are called grid north lines. Azimuths on the projection are measured relative to these grid north lines. When true meridians are projected onto the projection surface, only the central meridian will align exactly with these grid north lines. All other meridians will vary by some angle θ from the grid north lines. As geodetic azimuths are measured relative to meridians, the grid azimuth between any two points will be different than the geodetic azimuth between the same two points. An exception being if the two points both lie on the central meridian. Figure 5-2 shows this relationship.

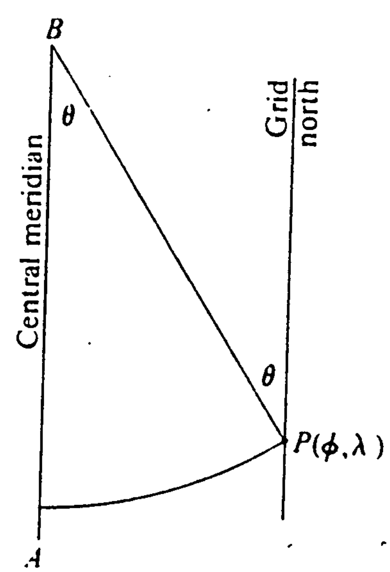

Figure 5-2. Relationship between geodetic and grid azimuths on Lambert Projection (9)

Point B is the apex of the projection cone. Line AB is the central meridian while the line PB is the meridian through P. The grid north line through P is also shown. The angle θ formed at B by the central meridian and the meridian through P is referred to as the mapping angle or angle of convergence at point P. This angle is a function of the difference in longitude between the central meridian and the point of concern. The mathematical relationship between geodetic and grid azimuth is given by:

$$\text{geodetic azimuth} - \text{grid azimuth} = +\theta.$$

The need often arises to convert back and forth between geodetic and grid azimuths when working in a state plane coordinate system. A slight correction to θ may be required if the line being converted was sighted between two points very far apart. A more detailed discussion of this correction and grid versus geodetic azimuth can be found in (9).

Coordinate Transformations

On a Lambert conformal projection, the central meridian is the Y axis with the X axis being perpendicular to the central meridian. The coordinate origin is far enough south and west such that positive X,Y values exist throughout the projection area. The Y axis is usually given a large X value, such as 2,000,000 (9).

In most applications using state plane coordinates, there exists a need to transform geographic latitude and longitude (ϕ,λ) values to state plane (X,Y) values. Throughout my project area, GPS provided the ϕ,λ values for points and the state plane X,Y values were subsequently computed. The following discussion shows how this transformation was done.

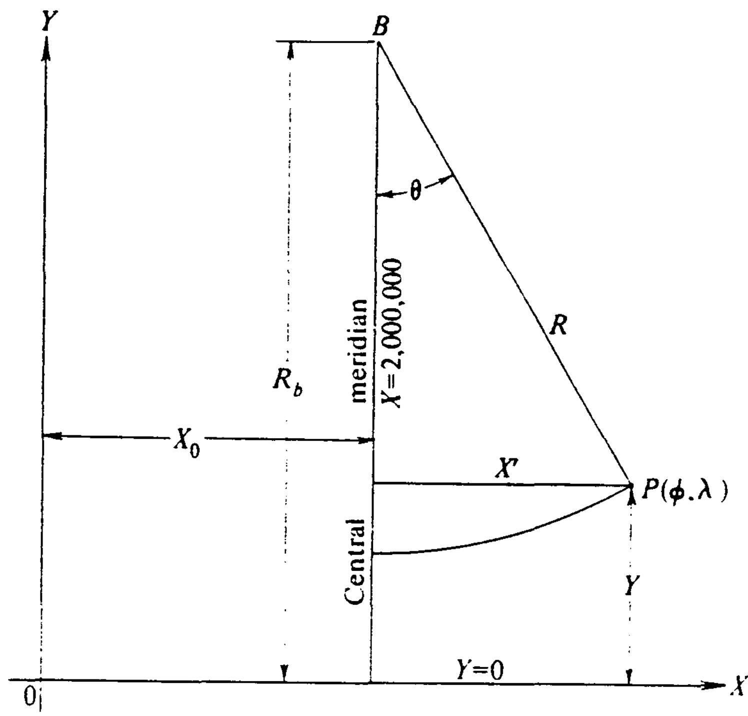

Figure 5-3. State plane coordinates on Lambert Projection (9)

Figure 5-3 shows a point P whose geographic coordinates, φ and λ, are known. The parallel of latitude through P is a circle of radius R. Point B is the apex of the cone of the projection, so R can be computed for any φ within a projection and tabulated. R_b is the distance from the X axis to point B along the central meridian and is constant for any projection. θ is the mapping angle at point P and is a function of the difference in longitude between the central meridian and P. The θ for each 1' of longitude has been tabulated for all projections. Once these values are determined, the X,Y coordinates for P can be computed from the following equation:

$$Y = R_b - R\cos\theta$$

$$X = X_o + R\sin\theta$$

I used a computer program developed by the National Geodetic Survey which incorporates the required tables and above techniques to do the transformation from NAD 83 ϕ, λ values to Iowa north zone X,Y values.

More discussion will be given to the techniques of reducing distances measured on the earth's topographic surface to its corresponding sea level distance and finally to its corresponding distance on the projection.

Distance Reductions

The first reduction is from the topographic surface to sea level. Figure 5-4 shows two points A and B on the topographic surface at an average elevation above sea level h. The line D-C represents the corresponding sea level distance between points A and B. This sea level distance is computed from the following formula.

$$D - C = D\left(1 - \frac{h}{R}\right)$$

where the term $(1 - \frac{h}{R})$ is called the sea level reduction factor.

R is the radius of curvature of the earth and is a function of the average latitude and geodetic azimuth between the two points. R can be assumed to equal 20,906,000 ft or be computed from

$$R = \frac{R_1 N}{R_1 \sin^2\alpha + N\cos^2\alpha}$$

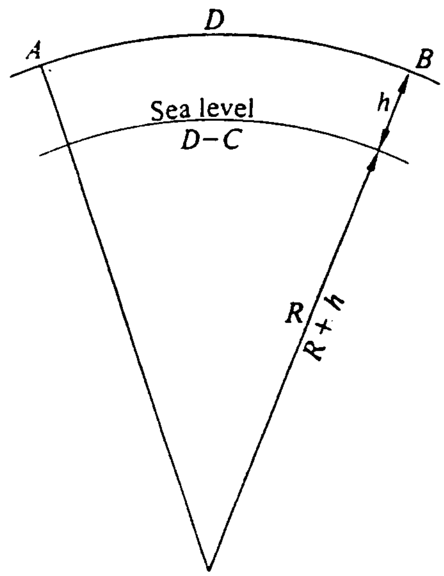

Figure 5-4. Reduction to sea level (9)

where

α = the geodetic azimuth between points

R_1 is the value of the radius of curvature in the plane of the meridian at the starting point of the line and is computed from

$$R_1 = \frac{a(1 - e^2)}{(1 - e^2 \sin^2\phi)^{3/2}}$$

a and e are parameters of the NAD 83 reference ellipsoid.

N is the value of the radius of curvature of the plane perpendicular to the meridian at the start point of the line. N is computed from the equation

$$N = \frac{a}{(1 - e^2 \sin\phi)^{1/2}}$$

Since most azimuths in both subdivisions ran generally north and south, the values computed for R_1 at control points 1 and 7 were used as the value for R in the computation of the sea level reduction factors for both subdivisions.

A more detailed discussion of these geodetic formulas is given in reference (5).

The second reduction is from the sea level surface to the projection surface. This is accomplished by multiplying the sea level distance times a grid scale factor. On a Lambert conformal projection, this value is a function of latitude. If the projection surface is below sea level at some ϕ, the scale factor will be less than one. If the projection surface is above sea level, the scale factor will be greater than one. Values for the grid scale factor have been calculated and tabulated for all projection surfaces.

It is common to combine the sea level reduction factor and grid scale factor into one to assist in computations. For my project, I developed one combined reduction factor for each subdivision which was applied to all topographic surface distances in the respective subdivisions. These factors were developed based upon the elevation and latitude for control points 1 and 7 as well as ellipsoid parameters for the NAD 83 ellipsoid. The grid scale reduction factor of the latitudes of these points was listed as part of the output from the program I used to convert the latitude and longitude values to state plane X,Y coordinates for each point. This one value was sufficient due to the flatness of the area as well as the small difference in latitude within each subdivision. The combined reduction factors for Britson's Second Addition was 0.999934014 and for Ryan's First Addition 0.999933575.

CHAPTER 6. COORDINATE COMPUTATION AND BASE MAP PREPARATION

Coordinate Computation

After the latitude, longitude, and geodetic height for each control point was computed from GPS surveying and accompanying computations, a computer program was utilized to transform the latitude and longitude values into state plane X,Y coordinates. This program follows the same algorithm for the transformation as presented in Chapter 5. At this point, the state plane coordinates for control points 1 and 7 had to be determined. Since the state plane coordinates of points 1A, 1B, 7A, and 7B were known, a surveying technique called intersection was used at both locations to determine the required coordinates.

Intersection is applicable in any situation where the position of a point is to be established but the point is not occupied (8). In Figure 6-1, point 1 was observed from points 1A and 1B. Angles a and b were measured using a directional theodolite and all distances between points were measured with an Electronic Distance Measuring (EDM)

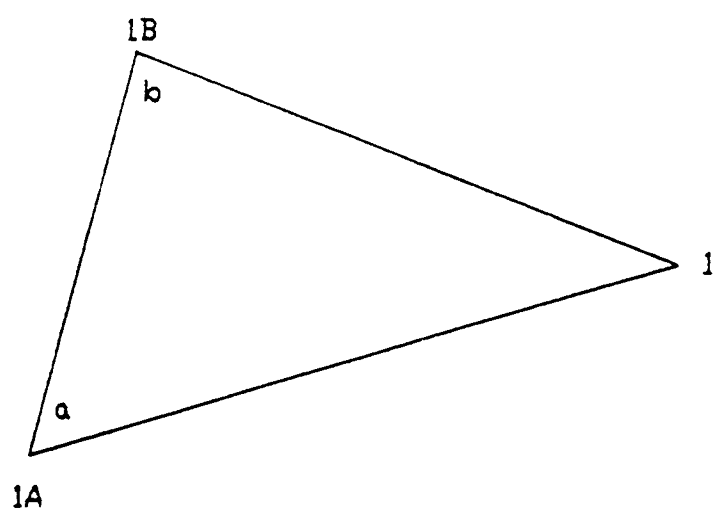

Figure 6-1. Intersection

device. These distances were then converted to projection distances. The azimuth of line 1A1B can be computed from the following equation:

$$Az_{1A1B} = ARCTAN\left(\frac{X_{1B} - X_{1A}}{Y_{1B} - Y_{1A}}\right)$$

and

$$Az_{1B1} = Az_{1A1B} + 180° - b$$

$$Az_{1A1} = Az_{1A1B} + a$$

Now, the values for X_1 and Y_1 can be computed as follows:

$$X_1 = X_{1B} + Distance\ 1B1(\sin\ (Az_{1B1}))$$

$$Y_1 = Y_{1B} + Distance\ 1B1(\cos\ (Az_{1B1}))$$

A check for blunders in the computations is made by computing the values of X_1 and Y_1 by using the above equation substituting the values on the right hand side of the equation with values for X_{1A}, Y_{1A}, distance 1A1, and Az_{1A1}.

Determination of the X and Y state plane coordinates for point 7 was done in a similar fashion.

The next computations were to determine horizontal distances and azimuths between control points aligning in north-south and east-west directions. This included similar computations from point 7 to point 6. The distance computations essentially served as a blunder check since the distances between 1/4 section corners and the section corner should be very close to 2,640 ft (1/2 mile). The four distances that could be checked in this manner all came very close to 2,640 ft with one exception.

The distance between points 4 and 7 was 2,655.14 ft. This large deviation was consistent with the distance computed between the same points by a local surveyor using an EDM for a different project. Resulting projection distances and azimuths between stations are shown in Figure 6-2.

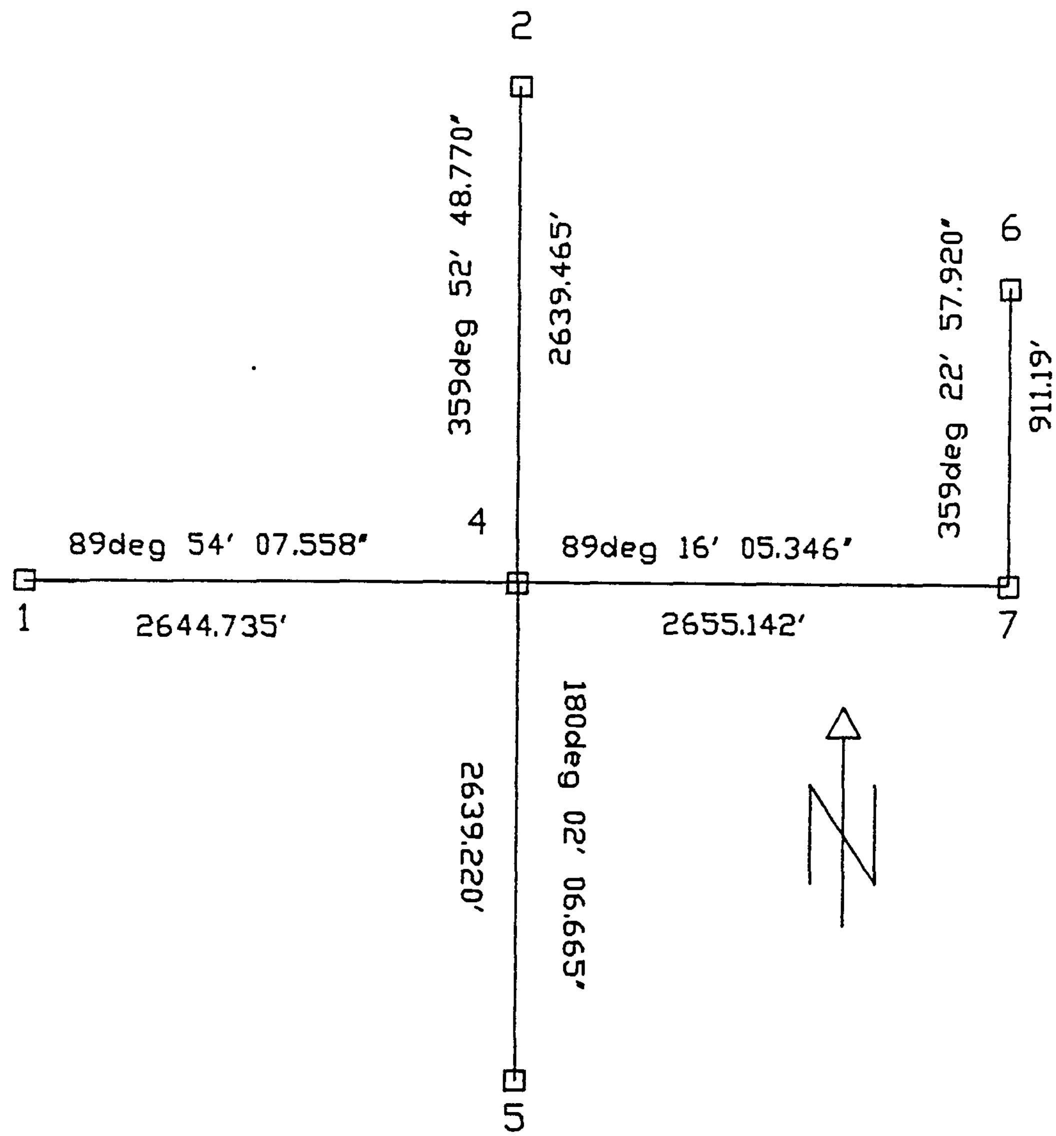

Figure 6-2. Projection azimuths and distances between key control points

The next step in the development of the base map was to determine coordinate values for lot corners and points along street centerlines in the two subdivisions. The coordinates could have been determined by locating all existing lot corner pins and completing a traverse in the field using traditional surveying techniques. A more practical approach is to use existing subdivision maps that have been prepared and filed as plats by local, professionally registered land surveyors. The key information I wanted from these maps was distance and bearings between consecutive points throughout the subdivision.

These subdivision maps are generally linked to a public land survey monument for ease of location. Such was the case for both subdivisions I worked with in Roland. A bearing and distance from this monument to the first point, or point of beginning, in a subdivision is given on the plat. As was shown earlier, if the X,Y coordinates of a point are known, and the azimuth and distance from that point to a second point are known, the X,Y coordinate of the second point can be readily calculated. Since the coordinates of the public land survey monument were determined, the coordinate of the point of beginning of the subdivision could be computed. The plat provides distance and bearings between consecutive lot corners in the subdivision, so the coordinates for all corners in the subdivision can also be obtained. Distance and bearing information is available to obtain centerline coordinates, also. Copies of the plats for both subdivisions are in Appendices A and B.

The distances and bearings from the plats had to be adjusted before they could actually be used in the coordinate computation. When a surveyor prepares a subdivision map, he uses a local coordinate system which is different than the state plane coordinate system. He assigns arbitrary coordinates to his origin point such as 1000, 1000. To establish direction, the surveyor will set his theodolite up on the origin point, sight on an object, and assign an azimuth to this line. All bearings determined will be relative to this initially established azimuth line. To translate these arbitrary bearings into bearings within the state plane coordinate system, at least one line where both the azimuth in the local coordinate system and state plane system are known is required. The difference in the azimuths is the angular difference in orientation of the coordinate axis of the two coordinate systems. This difference can be applied to any bearing in one coordinate system to find the equivalent bearing in the other coordinate system.

Figure 6-3 shows the common line for the Britson subdivision was the line from control point 1 to point 4. The difference of -00° 05' 07.6" was applied to all bearings listed on the plat to obtain the equivalent bearings in the state plane coordinate system.

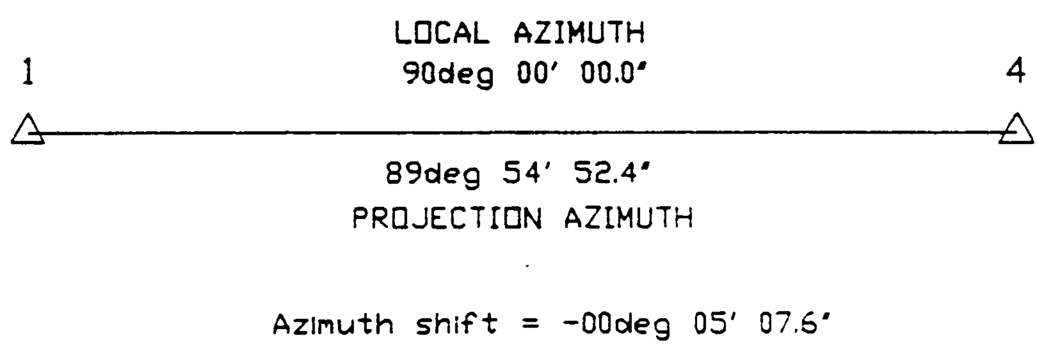

Figure 6-3. Common azimuth line in Britson's Second Addition

All distances listed on the plat were obviously measured on the surface of the earth and consequently were reduced to their projection distance through techniques described in Chapter 5.

The computation to convert distance and azimuths between consecutive points to coordinate values for the point is a repetitious execution of straightforward calculations. Because of this, I developed a Lotus 1-2-3 file to perform these computations for both subdivisions. As both subdivisions were primarily formed in a rectangular pattern, there were a limited number of different azimuth values within a subdivision. The bearings listed on the plats were manually translated to equivalent azimuths in the state plane coordinate system. The required input for the Lotus file was the X,Y state plane coordinates for the control point the subdivision was referenced to plus translated azimuths and surface distances between consecutive points. The file then computed the state plane X,Y coordinate for the required points. The coordinates for a small number of points in both subdivisions were manually computed as consecutive distances and bearings for the points were not provided on the plats. Appendix C shows the cell formulas for the Ryan Subdivision file.

State Plane coordinates computed following the above procedures represent locations on the state plane projection surface which has already been explained as not being the same as the surface of the earth. These locations are important for very precise, geodetic surveying applications. However, the concept of the projection surface being different than the surface of the earth and the associated concept that distance measured between the same two points is different on each surface are not commonly understood among land surveyors or the end users of most GIS. Therefore, the surface state plane coordinates were computed for the key points and these values were actually used to develop the base map. Although these values are not the absolute true state plane coordinate values, the confusion between projection versus surface values is eliminated.

The difference between surface and projection state plane coordinates was less than 1 part in 15,000. This is an acceptable accuracy for the application of most GIS. The projection coordinate values were also entered in the GIS database and are available to the user.

The surface state plane coordinates for key points in both subdivisions were computed by determining the surface coordinates for control points 1 and 7, then following the same procedure described to find the projection coordinates above, with one exception. When computing the surface state plane coordinates, the distances between points as shown on the plats were not reduced to their corresponding projection distances. They were used as shown on the plat. A Lotus 1-2-3 file was created to do the bulk of these computations also.

The surface coordinate values for control points 1 and 7 were computed using the projection coordinates for control point 4 as fixed. Next, the differences in the projection X,Y coordinates between point 4 and points 1 and 7 were divided by their respective reduction factors. These adjusted differences were then added or subtracted to the fixed coordinates at point 4 to find the surface state plane coordinates of points 1 and 7.

Base Map Preparation

The AutoCAD program was used to initially build the base map. It was felt this was a more versatile CAD environment than the IGS of UltiMap. The AutoCAD drawing files were then translated into the UltiMap program.

A separate map was created for each subdivision based in the surface state plane coordinate system. The drawing files for the maps were translated into UltiMap separately and then merged at a later time. Control point 4 was common to both maps which provided a check on the merging accuracy.

The maps were created at full scale which means one AutoCAD drawing unit equalled one foot.

The coordinates for each key point in the subdivision were entered into AutoCAD as points using the keyboard. These points were placed on one layer, and the lines connecting the points showing lot lines were placed on another layer. Centerlines and text were placed on separate, individual layers. This creation of more layers than might normally be used eases the translation process and will be explained further in Chapter 7.

CHAPTER 7. OTHER DATA ACQUISITION METHODS

When determining what information should be available in the GIS, it was determined that the location of houses, power poles, fire hydrants, and underground utilities should be included. It was also thought that topographic relief in the form of contour lines should be available. Photogrammetry was used to obtain the surface data and existing utility maps were used to obtain the underground information for Ryan's First Addition.

Photogrammetry

The photogrammetry work done for the project falls under the category of metric photogrammetry. Metric photogrammetry involves making precise measurements from photos and other sources to determine, in general, the relative location of points (15). I used a pair of overlapping diapositives of aerial photographs and a stereoscopic plotting instrument (or simply, plotter) in the project. This precisely built instrument projects light through diapositives to create a three dimensional stereomodel (or simply, model) of the overlap area. My work was done on the Kelsh plotter.

Figure 7-1 shows a typical direct optical projection stereoplotter very similar to the Kelsh plotter I used.

The key components are (1) the mainframe of the plotter; (2) the reference table, a large smooth surface upon which the manuscript map paper is placed and is also the vertical reference datum for models; (3) tracing table, which the platen and tracing pencil are attached; (4) platen, a flat surface upon which a portion of the model is illuminated which also contains the reference mark; (5) guide node, connected to the illuminating lamps so that projected rays will always fall on the platen, anywhere in the model; (6) projector; (7) illuminating lamps; (8) diapositives; (9) leveling screws, used to adjust the model in a process called absolute orientation; and (10) projector bar, to which the projectors are attached (15).

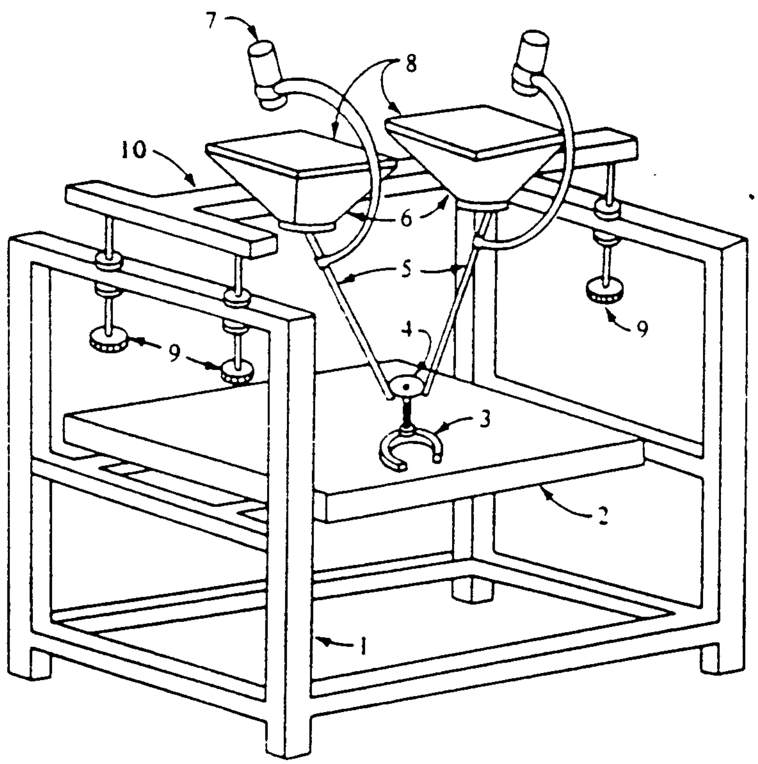

Figure 7-1. Components of a typical stereoplotter (15)

Figure 7-2 shows the basic concepts of stereoplotters. In Figure 7.2a, two successive aerial photos are taken with an overlap area as indicated. Diapositives are carefully prepared on glass or plastic plates from the negatives. The diapositives are placed into the projectors of the stereo plotter. Proper placement of a diapositives in the

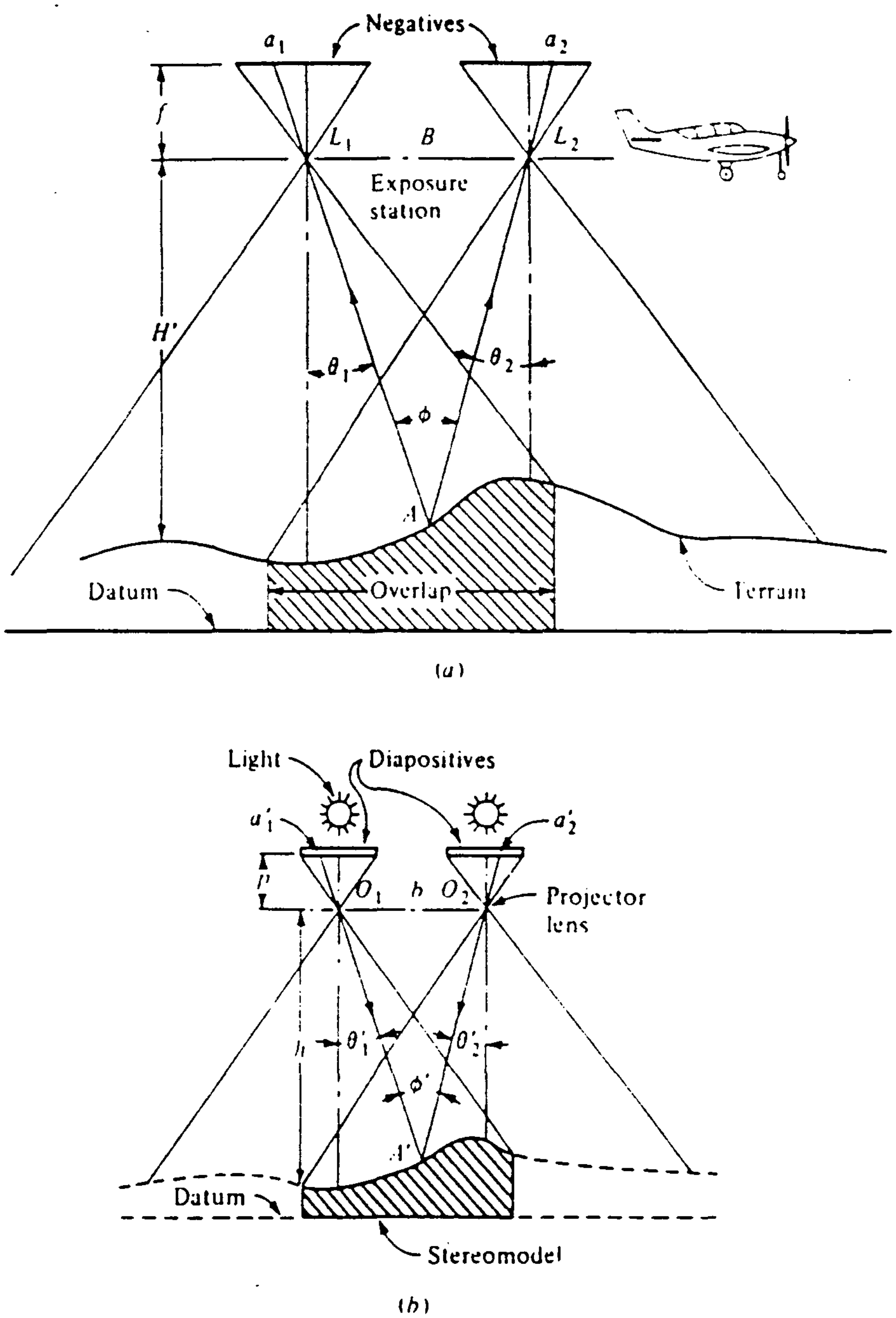

Figure 7-2. Basic concept of stereoscopic plotting instrument design. (a) Aerial photography; (b) stereoscopic plotting instrument (15)

projectors is called interior orientation. Next, light is shined through the diapositives, and the cameras are oriented such that light rays from corresponding images from the left and right diapositives intersect. This process is called relative orientation and places the diapositives in the exact angular orientation as the negatives were at the time of their exposure in the aircraft. After this, the model is adjusted to the desired scale and leveled with respect to a reference datum. Scaling and leveling are collectively called absolute orientation (15).

Measurements from the model can be made from the model once it is properly oriented. On the Kelsh, this is done graphically as shown in Figure 7-3. A reference mark in the center of the platen is brought in contact with a point in the model. This reference point is illuminated and is commonly referred to as the floating point. (As the platen is moved throughout the model, the illuminated reference point will appear to float above or below the terrain of the model). Points are plotted from the model onto the manuscript map with the use of a pencil located vertically below the floating mark. Elevations of points are read from a dial on the tracing table which moves with the up and down motion of the platen. This dial is calibrated during absolute orientation. Contour lines are traced by leaving the platen at one elevation and moving the platen around in the model keeping the floating mark in contact with the terrain (15).

Interior orientation essentially ensures the geometry of the diapositive with respect to the projector matches exactly the geometry of the negative with respect to the camera at the time of exposure. This means when placing the diapositives into the projector, the center of the diapositive must lie on the optical axis of the projector. This is accomplished slightly differently on different instruments. Interior orientation also involves setting up the

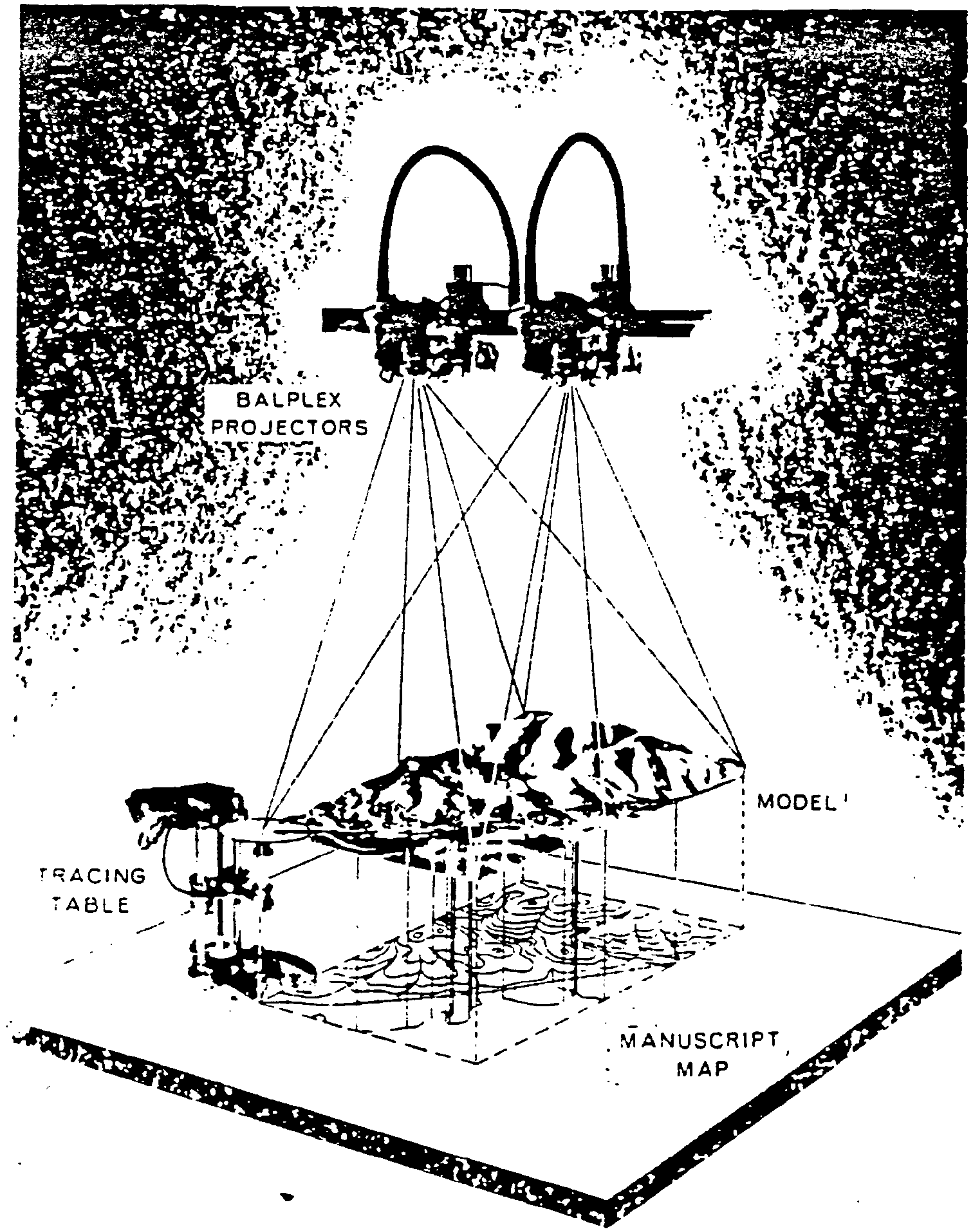

Figure 7-3. Typical stereomodel (15)

proper principal distance for the diapositive being used. Correct interior orientation is required so that angles θ_1 and θ_2 equal angles θ_1' and θ_2' in Figure 7-2.

Relative orientation is effected by six elements at each projector as shown in Figure 7-4. These elements are the translations in the X,Y,Z directions and the rotations about the X,Y,Z axis. Relative orientation is achieved by making mechanical adjustments to five of the possible twelve elements. Adjustments can be made to both or only one projector. These are known as double and single projector methods, respectively.

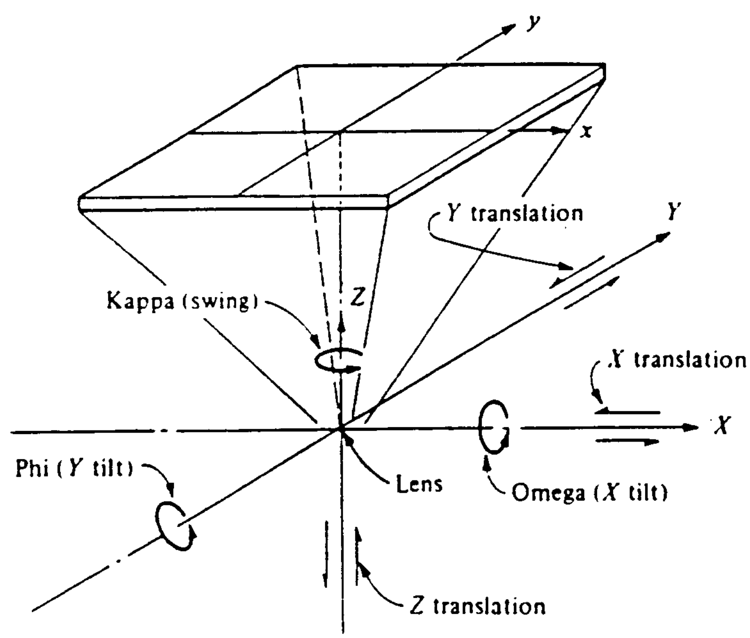

Figure 7-4. Six basic projector motions (15)

During relative orientation, adjustments are made to the orientational elements to remove y-parallax throughout the model. Figure 7-5a shows two projected images on a

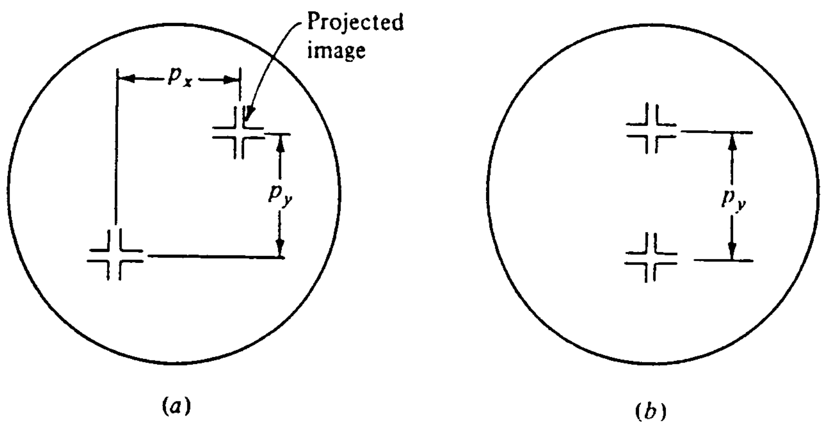

Figure 7-5. (a) Observing both y parralax and x parallax on the platen. (b) After removing x parallax, only y parallax remains (15)

platen in a model which is not yet properly oriented. The two images don't match because the light rays are not intersecting. The mismatch of the images consists of an x component and a y component, called x parallax and y parallax, respectively. The x parallax is a function of the elevation of the point and can be removed by raising or lowering the platen in the model. The y parallax is a function of improper orientation. This y parallax is removed by making adjustments to the projector's orientation elements at six different points in the model in a sequential, iterative process. When the y parallax is removed, or cleared, at all points, the model is relatively oriented.

Scaling and leveling the model requires a minimum of two horizontal and three vertical ground control points. For better results, a model will generally have four of each.

The distance between horizontal ground control stations is generally computed using their X,Y coordinate values by the following equation:

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

The desired scale of the model is reached by first plotting the known horizontal control points in the model. The distance between the points is measured and compared to the model distances that should be obtained at the desired scale. The required model distance for a particular ground distance at a given scale can be calculated from the following equation:

$$Model\ Scale\left(\frac{1\ inch}{X\ feet}\right) = \frac{Model\ Distance\ (inches)}{Ground\ Distance\ (feet)}$$

The model distance between control points is adjusted to match the required model distance by changing the distance between the two projectors. Bringing the projectors closer together makes the whole model smaller, thus reducing the scale of the model. Increasing the distance between the projectors has the opposite effect.

After relative orientation, the model will most likely not be level with the reference table as shown in Figure 7-6. Points A,B,C and D are the vertical control points with known ground elevation. The model is leveled by measuring the heights of the control points in the model and adjusting the model until the elevation differences between points in the model match the elevation differences on the ground. The level of the model can be adjusted with the rotational elements of the projectors or via leveling screws of the plotter as shown in Figure 7-7.

Relative orientation will usually need to be repeated after completion of absolute orientation. The model should then be checked for scale and level again. Once the model

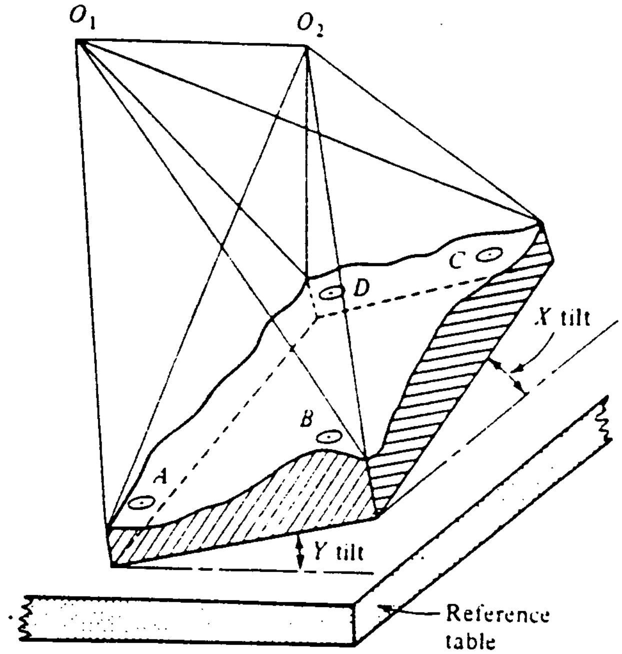

Figure 7-6. Stereomodel that is not level (note X and Y components of tilt) (15)

has been properly oriented, the final mapping manuscript is placed on the reference table. Ground control points should be preplotted onto this final manuscript, at the desired scale, so that these points on the manuscript can be aligned with the images of these points in the model. This orienting process ensures that points transferred from the model onto the manuscript will be plotted in the same coordinate system as the ground control points. More detailed instructions regarding stereomodel orientation can be found in (15).

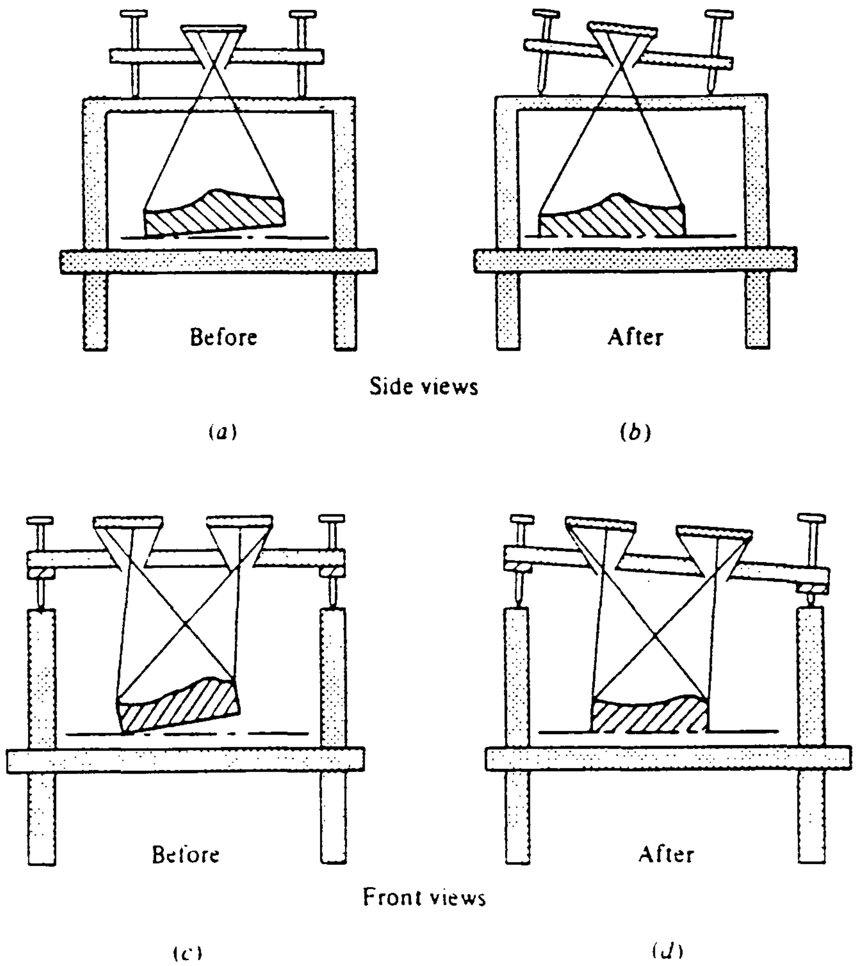

Figure 7-7. (a) and (b) Correcting X tilt of a model by X tilt of projector bar. (c) and (d) Correcting Y tilt of a model by Y tilt of projector bar (15)

A stereomodel of Ryan's First Addition was created following the procedures outlined above on a Kelsh stereoplotter. The outlines of the houses were plotted following the traces of the rooftops. Other information that was plotted were fire hydrants, power poles, and contour lines. The manuscript was created in the state plane surface coordinate system which allowed direct transfer from this manuscript into the already prepared base map. The photogrammetry information was digitized into the base map file using AutoCAD. Separate layers were created for each entity.

Existing aerial photography for the project was obtained from the Aerometrics Corporation. They had flown the photography for Story County as part of an earlier project. The photos were taken on March 3, 1988 at a scale of 1 in = 420 ft. On the Kelsh plotter, with a magnification factor of five, the model scale was 1 in = 85 ft.

The cost of existing photography is not inexpensive by itself, but presents a sizeable savings compared to having new photography flown. Existing photography is generally available through county and state agencies or private contractors.

Normally, ground control points are surveyed and marked prior to the photography being taken such that the points are easily seen in the model. Aerometrics had established ground control for the photos but the coordinate values were not available to us. This required us to establish our own ground control. The key in establishing ground control after the photography is taken is to select existing points that are imaged well in the model such as road or sidewalk intersections. Points 4 and 7 (see Figure 4-1) from the original survey were visible in the model and were used for ground control. Two additional ground control points were established using GPS surveying. These were points 3 and 8 in Figure 4-1. Figure 7-8 shows where the control points were located in the stereomodel developed to do the photogrammetry work. Figure 7-8 depicts two aerial photos with the shaded overlap area defining the model area. Having the control points at the four corners of the model eases the orientation process. Point 3 was established at a road intersection and point 8 at the last fencepost in a very visible fence row.

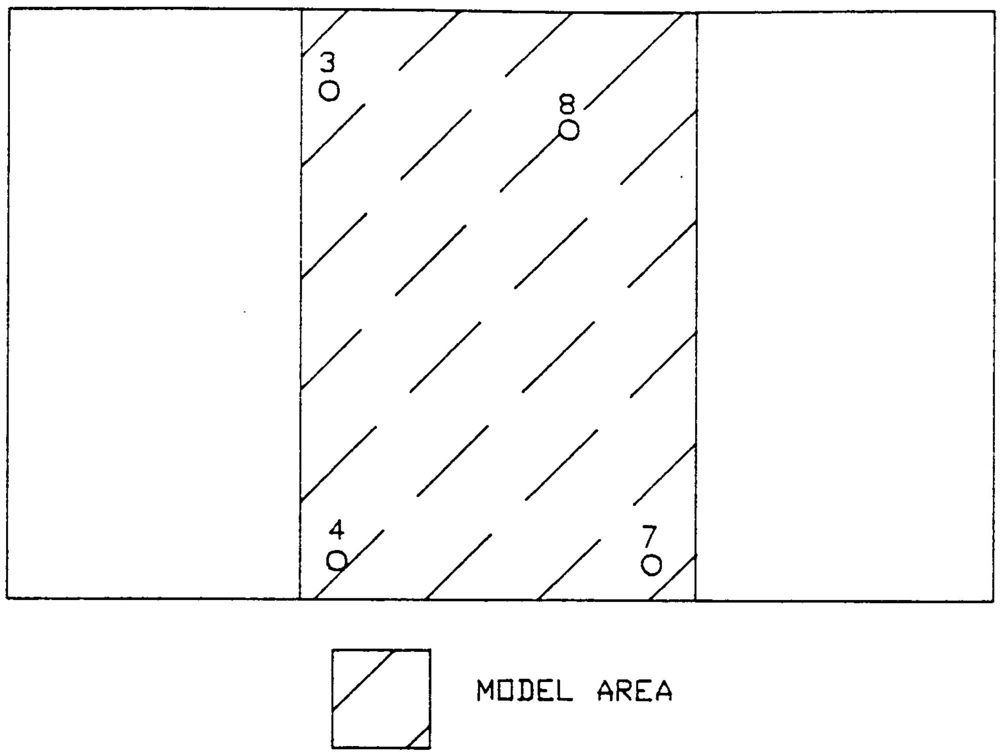

Figure 7-8. Control points for Roland stereomodel

Stereoplotters called analytical stereoplotters eliminate the digitizing step for data entry. Analytical stereoplotters electronically transfer the coordinates of entities as they are plotted into a digital base map. This type of plotter was not available for this project, but direct coordinate computation can be accomplished using the Zeiss C-8 stereoplotter. This technique provides an alternative method of data entry to the use of the Kelsh and subsequent digitizing.

The Zeiss C-8 stereoplotter has an optical-mechanical projection system which causes the viewer to see the model through a set of binoculars on the machine. Another major feature of the Zeiss C-8 is that it provides an analog read out of X,Y,Z model coordinates for any point in the model. By establishing a mathematical relationship between the model coordinates and ground coordinates, the ground coordinates for any point in the model can be readily computed. These coordinate values could then be directly entered into the GIS map to locate an entity.

The relationship between the model coordinates and ground coordinate system is a transformation between the coordinate systems. The transformation parameters are computed using the model and ground coordinates for the control points.

The basic transformation equations are:

$$X_G = (S)X_m + b_1Y_m + C_1Z_m + X_o$$

$$Y_G = (S)Y_m + b_2X_m + C_2Z_m + Y_o$$

$$Z_G = (S)Z_m + b_3X_m + C_3Y_m + Z_o$$

where:

X_G, Y_G, Z_G are the ground coordinates of a point;

S is the scale factor of the model;

X_m, Y_m, Z_m are the model coordinates of a point;

b_1, b_2, b_3, C_1, C_2, C_3 are the unknown rotational transformation parameters; and

X_o, Y_o, Z_o are unknown shift components in the coordinate system origins (translations)

Three equations of this form can be written for each control point. The known values are substituted in and the equations are rewritten into the following format:

$$b_1Y_m + C_1Z_m + X_o = X_G - S(X_m)$$

$$b_2X_m + C_2Z_m + Y_o = Y_G - S(Y_m)$$

$$b_3X_m + C_3Y_m + Z_o = Z_G - S(Z_m)$$

All equations for the control points are listed, and then rewritten into matrix form of $Ax = \ell$ where A is the coefficient matrix of the left hand side of the equations, x is the matrix of unknowns, and ℓ is the matrix of the values on the right hand side of the equations. If three control points are used, A will be a 9 x 9 matrix, x will be a 9 x 1 matrix and ℓ will be a 9 x 1 matrix. The matrix x of unknowns is solved using least squares procedures such that

$$x = (A^TA)^{-1}A^Tl$$

Once the transformation parameters are known, they can be applied to any point in the model to determine the corresponding ground coordinates for that point.

Utility Plan and Profile Drawings

Underground utility data were gathered from existing plan and profile drawings for water supply and sanitary sewer lines.

The plan drawings were used to digitize the location of the utility lines onto the existing base map. The plan drawings are scaled drawings and were oriented to the existing base map for digitizing by using points common to both the plan drawing and existing base map as set-up points. Appendix D shows the plan drawing used for the water supply line.

The profile drawings showed key attribute information for the utilities. Attribute data available were minimum depth of fill over the utilities, % slope, type, and diameter of pipe. The data were combined with location data for power poles and fire hydrants into one utility attribute data file which can be accessed through different UltiMap procedures described in Chapter 8. Appendix E shows the water profile drawing used.

CHAPTER 8. DEVELOPING THE ROLAND GIS APPLICATIONS

There were two general capabilities I wanted to provide with the Roland GIS. First, I wanted to provide the capability to store, retrieve, and display attribute data pertaining to graphical entities. More specifically in this regard, I wanted to develop the application to be viewing a map of the city, select an entity with the cursor, and have the system display attribute data pertaining to that feature. An example of this would be for the user to be able to select a lot within a subdivision and have the system display who owns the lot and what the street address is.

Secondly, I wanted to provide the capability to analyze an existing attribute data file and indicate all graphical features which met a particular attribute description provided by the user. An example of this application would be for the user to be viewing a map of a subdivision and be able to instruct the system to highlight all the lots with an area greater than 10,000 square feet.

The specific attributes available for processing differed between the two subdivisions of the project, representing the needs of different users of the system. The information available in Britson's Second Addition would be of primary use to those concerned with boundary location and control while Ryan's First Addition data provides information useful for more expanded areas of municipal engineering, tax assessment, and property purchasing. Certainly, both sets of attributes could be developed for both subdivisions.

Specific attributes available for processing within Britson's Second Addition are:

1. Owner of each lot in the subdivision.
2. The street address for each lot in the subdivision.
3. The area and perimeter of each lot in sq. ft. and ft., respectively.

4. The surface and projection state plane coordinate for all lot corners.

5. The bearings of all lot lines in the subdivision.

6. The current assessed value of each lot in the subdivision.

The following attributes are available for processing within Ryan's First Addition:

1. The owner of each lot in the subdivision.

2. The street address of each lot in the subdivision.

3. The number of bedrooms in each home in the subdivision.

4. The surface state plane coordinates for the fire hydrants and power poles.

5. The type of pipe, diameter of pipe, average % of slope, and minimum depth of coverage for sewer and water pipes.

The map of this subdivision also contains more graphical features than does the map of Britson's Second Addition. These additional features are the outlines of the houses, sanitary sewer lines, water lines, contour lines, fire hydrants, and power poles. As was mentioned earlier in the paper, these features were added to the base map from photogrammetry work and existing plan and profile drawings.

The majority of the graphics work was done in AutoCAD. The DXF files for the AutoCAD drawings were then transferred from floppy disk to the hard drive of the home directory of the Apollo Network. This transfer was accomplished using a File Transfer Program (FTP) available on the Iowa State Network (ISN).

Once the DXF file was in the Apollo home directory, the UltiMap translator routine was invoked to translate the AutoCAD DXF file into an UltiMap drawing file in the IGS module. During this translation, each AutoCAD drawing layer must be assigned an UltiMap symbol definition. Also during the translation, the user can specify the drawing scale and the minimum starting reference number to assign to the different entities in the drawing. During the translation, each entity in the AutoCAD drawing will be assigned an UltiMap reference number in the order in which the entities were drawn. Because of these translation actions, like entities should be drawn on the same layer and in consecutive order in AutoCAD. A two-point transformation is automatically done during the translation to ensure proper orientation and scale of the UltiMap drawing.

Each subdivision drawing was translated into IGS with a scale of 1 in. = 100 ft. Miscellaneous touch-up work was done in IGS such as adding some street boundaries and text. In addition, polygon areas were created out of all the lots in Britson's Second Addition and the house outlines in Ryan's First Addition.

The next step was to create the nongraphical data base. This consisted of using the Apollo editor to create ASCII files with keyboard entry. These files would then be converted to a machine readable format using the Q-join module. The ASCII files were created in the Apollo Variable Length Format which allows 256 columns for each record. Two hundred fifty-six columns allows for the listing of many attributes for an entity in a single record which eases file construction.

A separate data file was built for each entity that attribute data would be tied to. For example, in Britson's Second Addition, one data file was built for attribute data to be linked to the lots, one data file was built for attributes tied to the lot lines, and one data file was built for attributes linked to the lot corners. For Ryan's First Addition, one data file was built for attribute data linked to the houses in the subdivision and one file for attribute data linked to the utilities. Separate files for like type entities were built because common entities share nearly identical attributes. Within the files, each record contained all attribute data pertaining to an individual entity. Within each record, fields were assigned to carry attribute values.

The files were created and then defined in the Q-link submodule. The ASCII files were converted to a specific UltiMap format called an UltiMap Index File Structure (UMIFS) using the Q-join submodule. The first field in each file was defined as the index field which would be used in subsequent processes.

The final stage of building the GIS was to use the TAG and Q-view submodule of the program to link the graphics with the nongraphical data.

The TAG module was used to link reference numbers of graphic entities with specific values in the index fields of the data files. A separate tag scheme was created for each non-graphic data file. Reference numbers for individual entities were retrieved from the system through the IGS module. These reference numbers were then individually linked with specific index field values through the various TAG module processes.

The Q-view submodule was then used to instruct the system on how to utilize the attribute data files and TAG schemes to achieve the desired applications. Each set of instructions for a particular process is called, simply enough, a Q-view.

One Q-view for each attribute file was written such that when a particular item was located with the cursor while in the IGS module, all attributes pertaining to that feature would be displayed on the screen. What attributes are displayed, where they are displayed within the display window, and even the font style of the display are defined within the Q-view. With a minor modification, this output can be sent to a file, and then the file can be printed for a hard copy report. Figure 8-1 shows an example of one such report for Britson's Second Addition. Other report examples are found in Appendices F through I.

LOT #	SUBDIVISION	OWNER	AREA (SQ. FT.)
LOTNO02	BRITSON'S 2ND ADDITION	STEVE TWEDT	11034.68

ADDRESS	CURRENT ASSESSED VALUE	PERIMETER (FT)
110 BRITSON CIRCLE	54300	425.22

SURFACE STATE PLANE COORDINATES OF LOT CORNERS
STARTING WITH NW CORNER OF LOT AND PROCEEDING CLOCKWISE:

	CORNER 1	CORNER 2	CORNER 3
NORTHING (FT)	3523503.023	3523503.205	3523413.205
EASTING (FT)	4918693.724	4918816.303	4918816.319

	CORNER 4	CORNER 5	CORNER 6
NORTHING (FT)	3523413.023	XXXXXX	XXXXX
EASTING (FT)	4918693.682	XXXXX	XXXXX

Figure 8-1. Q-view report for Britson's Second Addition lot information.

Other Q-views were written allowing a user, while in IGS, to have the system highlight a particular entity or entities which matched a logical expression he provided with keyboard input. One such Q-view was written for Britson's Second Addition and two were written for Ryan's First Addition. These Q-views were also designed to display a limited amount of attribute data on the screen.

The RCDMS module was then used to load the separate subdivision drawings to the UltiMap database, merge them, and load the merged map into a third drawing file at a scale of 1 in. = 200 ft. All Q-views can be invoked in IGS whether the user is working with the drawing file with the subdivisions merged or a drawing file of an individual subdivision. Borders and legends were then added to each drawing so they could be plotted as individual maps in the plotting process within IGS. Appendix J shows a copy of the map of both subdivisions at a scale of 1 in = 200 ft.

Figure 8-2 shows a flow chart for the use of the UltiMap program to build a GIS.

TASK	APOLLO/UltiMap FUNCTION
PREPARE GRAPHICS	IGS/TRANSLATOR/RCDMS
BUILD ASCII ATTRIBUTE DATA FILES	APOLLO EDITOR/Q-LINK
CONVERT ASCII FILES TO UMIFS FORMAT	Q-JOIN
CREATE TAG SCHEMES	TAG
INSTRUCT SYSTEM TO ACCOMPLISH INDIVIDUAL APPLICATIONS	Q-VIEW

Figure 8-2. Flow chart for use of UltiMap program to build a GIS.

CHAPTER 9: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

A working GIS was successfully developed for two subdivisions of Roland. The system accomplishes the goal of providing the user with information necessary to make decisions for management purposes. An automated GIS could benefit Roland by increasing the efficiency of information processing and by providing a permanent computer record of data.

The user now has at his fingertips information that was scattered in three different offices in two different towns located 13 miles apart. Simple information such as who owns a specific lot in a specific subdivision is now available with a few keyboard strokes instead of a 15-30 minute search through the subdivision plats and a phone call, during office hours only, to the county assessor.

In addition to the increased efficiency an automated GIS could offer the City of Roland, it would also provide a permanent record of all stored data. This information would still be available if a key employee was not available or if a map or other document was inadvertently thrown away or misplaced in the map box that is "kind of organized" in a second floor room of City Hall.

Feedback from a presentation made to the city clerk confirmed these conclusions. She felt the developed applications would be very useful to Roland. Her strongest feeling was that the applications developed would make data retrieval much more efficient than it currently is. She thought a GIS could be especially useful in a current housing development project by being able to identify all vacant lots in town along with their zoning and utility information. The clerk also thought a GIS would be very beneficial to newcomers, such as herself, in a small town environment where much institutional knowledge exists only in the heads of experienced employees. Although the clerk was supportive of an automated GIS, cost of the system is the key issue. The UltiMap/Apollo system would definitely be cost prohibitive. Roland has a development corporation that could provide funds for a GIS, but a detailed cost/benefit study would be required to obtain those funds. It should be pointed out the intent of the project was not to prove the UltiMap/Apollo system would be cost effective for Roland. Roland was selected solely because of the benefits it provided the project as listed in Chapter 2.

A GIS is a powerful tool for a decision-maker in its finished form. A key point I learned from this project is the complexity of a GIS and that it is not an overnight venture to develop a GIS to its finished form. A tremendous amount of knowledge and work goes into a GIS before the user can sit down at a terminal, place the cursor on a feature displayed on the screen, and click the mouse button to have attribute information displayed about that entity. My biggest lesson learned regarding time required to accomplish a task was entering nongraphical attribute data into the system. It took about twice as long as expected to design input and output file formats as well as do the actual keyboard entering of the attribute data.

Time can be saved if the system you are using has the ability accept already existing data files such as DIME files. Obviously, you have to have a need for these existing data files, but time can be saved if your system can accept existing data files which match your needs. The UltiMap program accepts many data file formats and saved me considerable time through its ability to translate AutoCAD DXF files into its own graphics subroutine. This compatibility feature should be a key feature looked for when purchasing software and hardware.

Having a firm idea of what the final product of your efforts should be is as important in developing a GIS as is any engineering or scientific project. Knowing what graphical features you want available for display is needed so you can organize your drawing layers. Knowing what features you are going to attach attribute data to is needed so you can organize how these features will be linked to the attribute data base. You need to know what attribute you want attached to your graphical features to organize how the data files will be constructed. The list of "need to knows" is very long, but the point is, you need to plan ahead when developing a GIS.

A coordinate base map for the GIS was developed. Specific techniques involved in the creation of the map are listed below:

1. Collect subdivision plats and utility plans for areas to be mapped.
2. Use GPS surveying and NGS software to to establish ground control in State Plane Coordinate System.
3. Rotate bearings of subdivision plats into state plane projection azimuths.
4. Compute coordinates of lot corners and centerline points using Lotus 1-2-3.
5. Enter coordinates into AutoCAD on one drawing layer.
6. Connect the points showing lot boundaries on a separate layer.
7. Add text.
8. Digitize features gathered from photogrammetry. Put all features on separate layers.
9. Digitize underground utility information from utility plan on separate layers.

10. Translate AutoCAD files into UltiMap.
    a. Assign each AutoCAD layer an UltiMap dictionary symbol.
    b. Assign desired scale. UltiMap equates one AutoCAD drawing unit to one inch.
11. Do final editing within IGS module.

The state plane coordinate system is an appropriate coordinate system to use for a GIS base map. Using the state plane coordinate system allows for compatibility between base maps at the city, regional, county and state level. Use of GPS and commonly available transformation software makes the establishment of state plane coordinates for control points and resulting baseline information a relatively simple operation.

The use of GPS surveying techniques to assist in the development of the base map as well as establishing ground control for the photogrammetry work was very successful. I established the location of 10 points in the project including offset and photogrammetry ground control points. GPS techniques allowed location of these points with centimeter accuracy. The field and office work to determine the locations for these points took about 14 hours. This is comparable to the time it would take an experienced conventional survey crew to do the same work once they were in the vicinity of the project. However, the traditional survey crew would incur additional time in running a traverse from some distant geodetic control monument with a known latitude, longitude, and elevation. This would be needed to establish a starting control point at the local work area in Roland. GPS surveying establishes latitude, longitude, and elevation for any occupied point independent of any other point.

Photogrammetry is an invaluable asset in the development of a GIS with applications similar to my project. It simply provides information that is not available from any other source and generally provides more current information than other sources. When the photography of the area of interest becomes available, the procedures to capture the data are fairly rapid and can be entered into the GIS accurately. Much more information than I used in my project is available for input to a GIS from photogrammetry. Locations of individual or groups of trees could be gathered. The City of Newark, NJ used photogrammetry to locate such details as manhole covers and emergency telephones (6). Photogrammetry is expensive, but should always be considered when gathering data for a GIS application similar to the Roland project.

Recommendations

The following recommendations are made resulting from the completion of my project.

1. GPS surveying techniques continue to be utilized in the development of GIS base maps.
2. Coordinate maps may be developed as base maps for GIS projects whenever practical.
3. The use of photogrammetry continues to be used as a data source for GIS.
4. Follow-up research to my project could be conducted in the following areas:
    a) Develop a similar GIS of Roland on a PC-based GIS followed by a cost analysis to determine affordability of such a system for Roland and cities of similar size and budget.
    b) Mapping and including the whole city of Roland in a GIS.

c) Developing a GIS for Story County incorporating the data I have collected for Roland as well as similar data for all other cities in the county.

5. Upon receipt of better documentation, which is currently being prepared for distribution, I'd recommend the UltiMap program continue to be utilized for GIS research at ISU.

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I also express my appreciation to the United States Army for providing me the opportunity to pursue graduate studies at ISU.

I thank my family for their patience and support throughout the past two years.

APPENDIX A: RYAN'S FIRST ADDITION PLAT

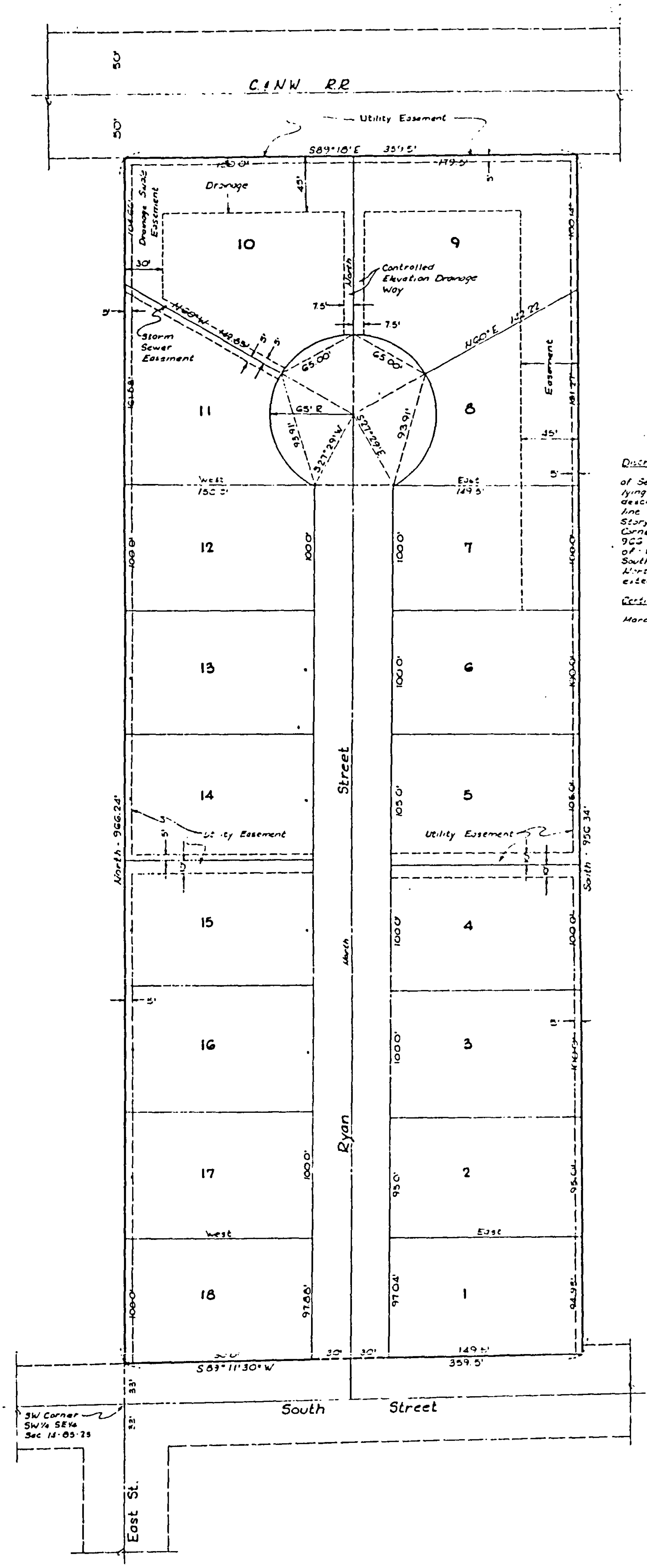
FINAL PLAT

RYANS 1ST ADDITION

CITY OF ROLAND, IOWA

NORTH

Scale 1" = 50'



Discription:

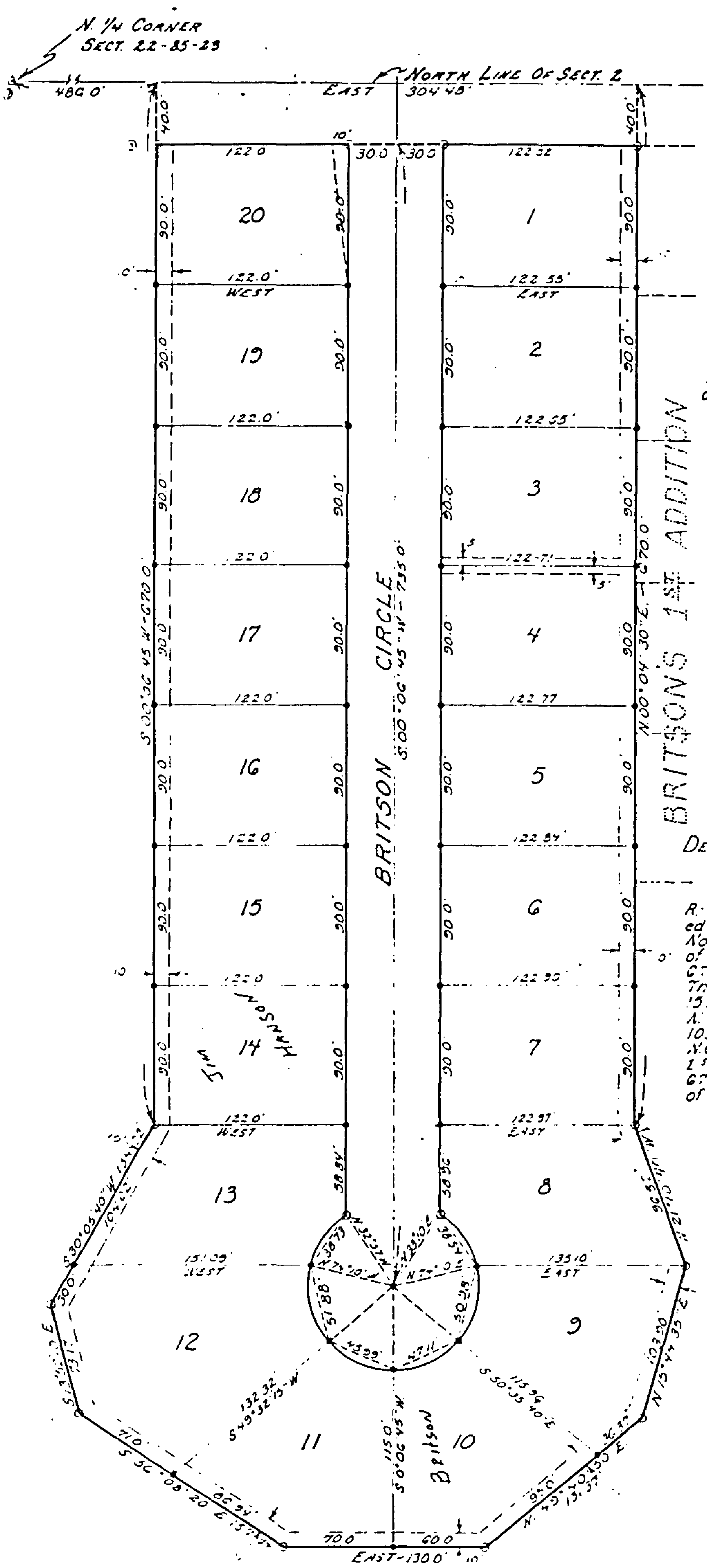
The west 21 rods and 15 feet of that part of SW¼ SE¼ of Sec 14 - Twp 85N R23W of 5th PM, Story County, Iowa lying southerly of the C & N W ROW, more particularly described as follows: "Beginning at a point on the West line of SE¼ of Sec 14 Twp 85N - R23W of the 5th PM, Story County, Iowa, which is 330 feet North of the SW Corner there of; thence North along said West line, 966.24 feet; thence, S89°18'E, along the south Right-of-Way line of C & N W RR, 359.5 feet; thence South, 956.34 feet; thence S89°11'30"W along the North Right-of-Way line of a County road (South Street extended East) 359.5 feet to the point of beginning."

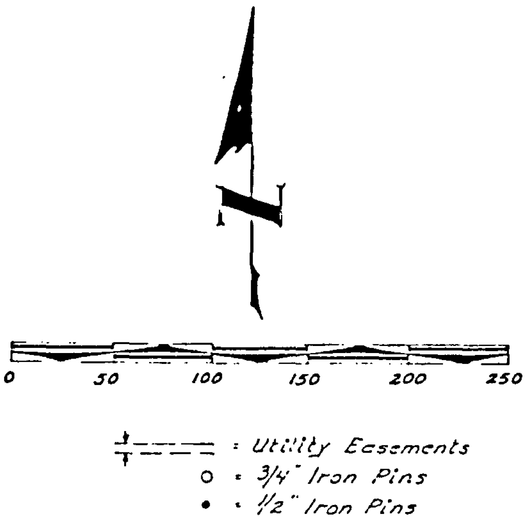
Certification:

I hereby certify this to be a plat of a survey made in March, 1976 for Tommy and Sylvia Ryan, Roland, Iowa

James M Burns
PE & LS #2622

APPENDIX B: BRITSON'S SECOND ADDITION PLAT





DESCRIPTION:

A part of the N.E. 1/4 of Section 22 - Twp 85 N.-R. 23 W of the 5TH P.M., Story County, Iowa, described as follows: "Beginning at a point on the North line of said N.E. 1/4 which is 486.0 feet East of the N 1/4 Corner thereof; Thence S 00°06'45" W, 670.0 feet; Thence S 30°05'40" W, 134.02 feet; Thence S 13°52'10" E, 79.17 feet; Thence S 56°08'20" E, 157.94 feet; Thence East, 130.0 feet; Thence N 49°40'50" E, 131.37 feet; Thence N 15°44'35" E, 103.90 feet; Thence N 21°10'40" W, 96.52 feet; Thence N 00°04'30" E, along the West line of Britson's 1ST Addition, extended South, and that line itself, 670.0 feet; Thence West, 304.45 feet to the point of beginning.

CERTIFICATION:

I hereby certify that the above plat is in accordance with the survey made in March 1973 for Bette Ritland, E. G. Ritland, Ralph A. Britson, Rosemary Britson, Loran K Britson, Patricia Lee Britson, William Britson, Hope Britson, William Maynard and Dolores Maynard.

James M. Burns
JAMES M BURNS
R.L.S. # 2622

JAMES M. BURNS REGISTERED 2622 IOWA PROF. ENGR. & LAND SURVEYOR

– FINAL PLAT –
OF
BRITSON'S SECOND ADDITION
TO
ROLAND, IOWA

The Northerly 40 feet of the above tract and that part shown as "Britson Circle" are hereby dedicated as public streets.

·March 1973·

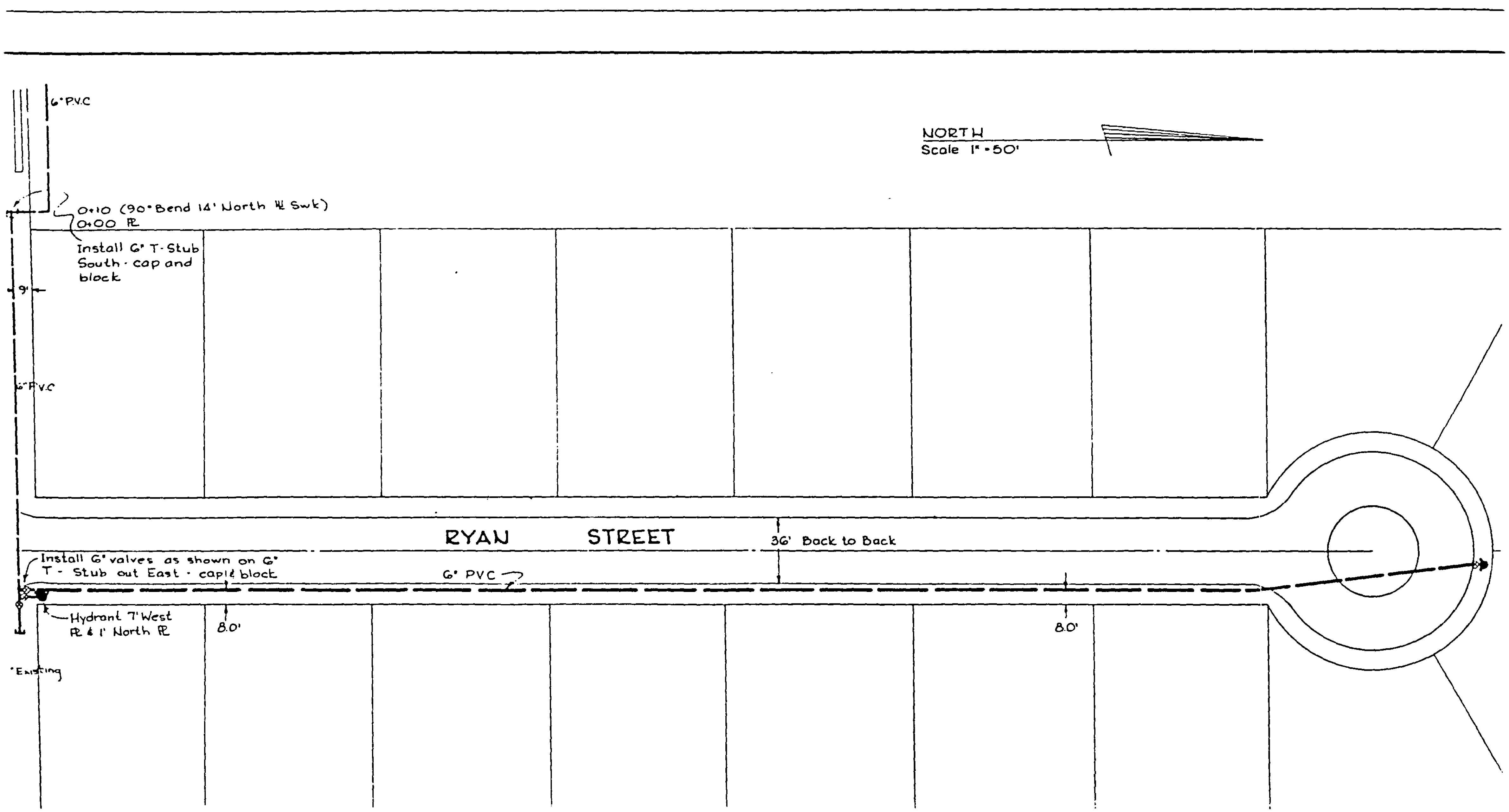
·Ames Engineering & Testing Company – Ames, Iowa·

APPENDIX C: LOTUS 1-2-3 CELL FORMULAS FOR COORDINATE COMPUTATIONS

```
A3:   ^1
C3:   [W14] 0.990034014
G3:   ^1
J3:   ^1
K3:   [W15] 1499032.544
L3:   [W15] 1074016.097
M3:   [W15] +K3*3.2808
N3:   [W15] +L3*3.2808


A4:   ^2
B4:   486
D4:   [W14] +B4*$C$3/3.2808
E4:   [215] 89.91455556
F4:   [W9] +E4*0.017453293
G4:   ^2
H4:   [W10] @COS(F4)*D4
I4:   [W11] @SIN(F4)*D4
J4:   ^2
K4:   [W15] +I4+K3
L4:   [W15] +H4+L3
M4:   [W15] +K4*3.2808
N4:   [W15] +L4*3.2808
```

APPENDIX D: WATER PLAN DRAWING FOR RYAN'S FIRST ADDITION


6" P.V.C
NORTH
Scale 1" = 50'
0+10 (90° Bend 14' North ℄ Swk)
0+00 ℄
Install 6" T-Stub
South - cap and
block
9'
6" P.V.C
RYAN STREET
36' Back to Back
Install 6" valves as shown on 6"
T - Stub out East - cap & block
6" PVC
Hydrant 7' West
℄ & 1' North ℄
8.0'
8.0'
Existing

APPENDIX E: WATER PROFILE DRAWING FOR RYAN'S FIRST ADDITION

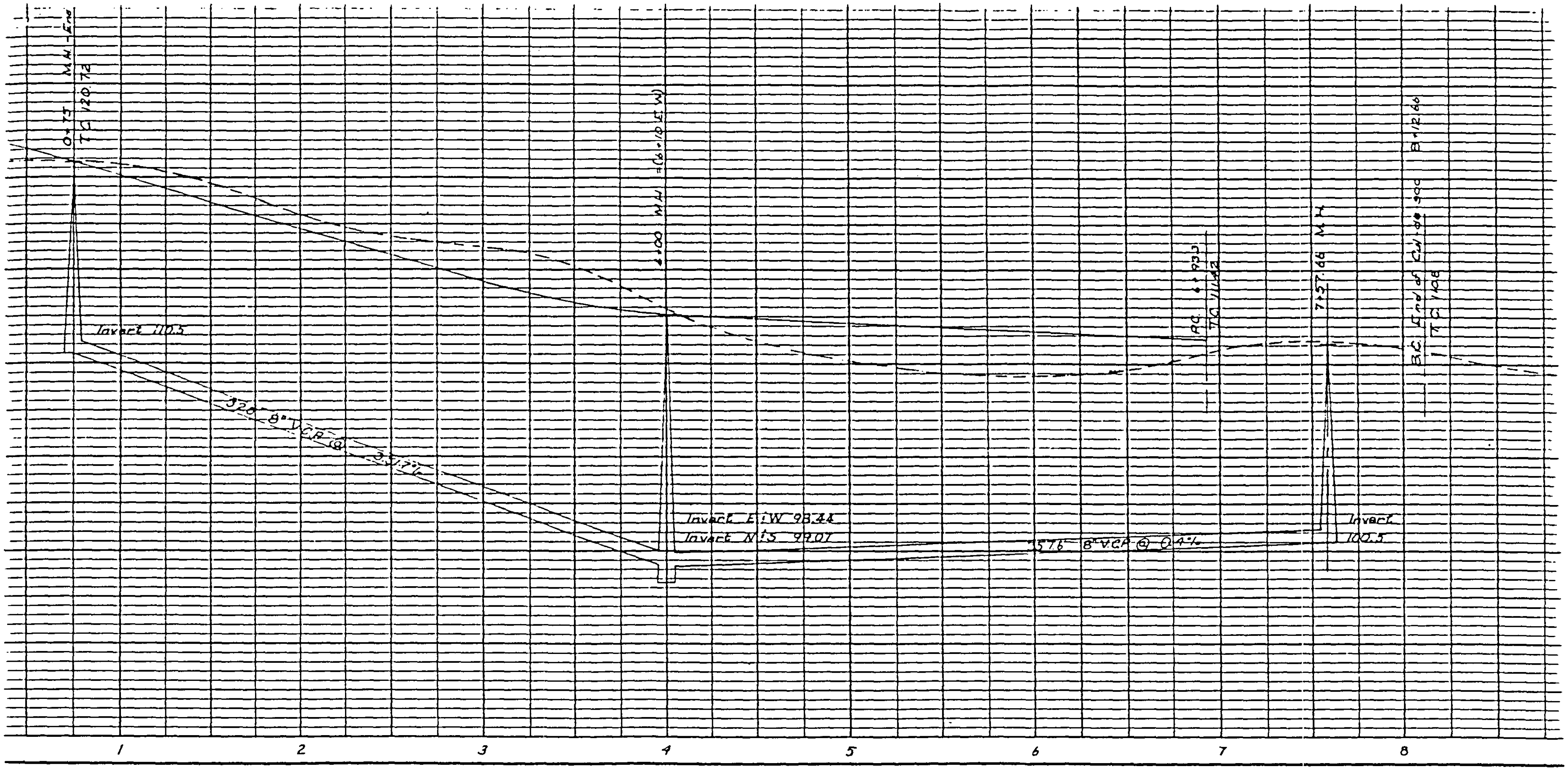


PLATE 1 PLAN-PROFILE B P R STANDARD
EUGENE DIETZGEN CO., CHICAGO - NEW YORK

APPENDIX F: UTILITY Q-VIEW REPORT FOR RYAN'S FIRST ADDITION

UTILITY	PIPE DESCRIPTION	MINIMUM DEPTH OF FILL
SANITARY SEWER	8IN VCP	10.5 FT

AVERAGE SLOPE	LOCATION (SURFACE STATE PLANE COORDINATES)	
2%		
	NORTHING	EASTING
	XXX	XXX

APPENDIX G: HOME Q-VIEW REPORT FOR RYAN'S FIRST ADDITION

ADDRESS	OWNER
201 RYAN STREET	CHARLES WRIGHT

NUMBER OF BEDROOMS IN HOME	YEAR HOME WAS BUILT
3	1977

APPENDIX H: LOT CORNER Q-VIEW FOR BRITSON'S SECOND ADDITION

PROJECTION STATE PLANE COORDINATES:

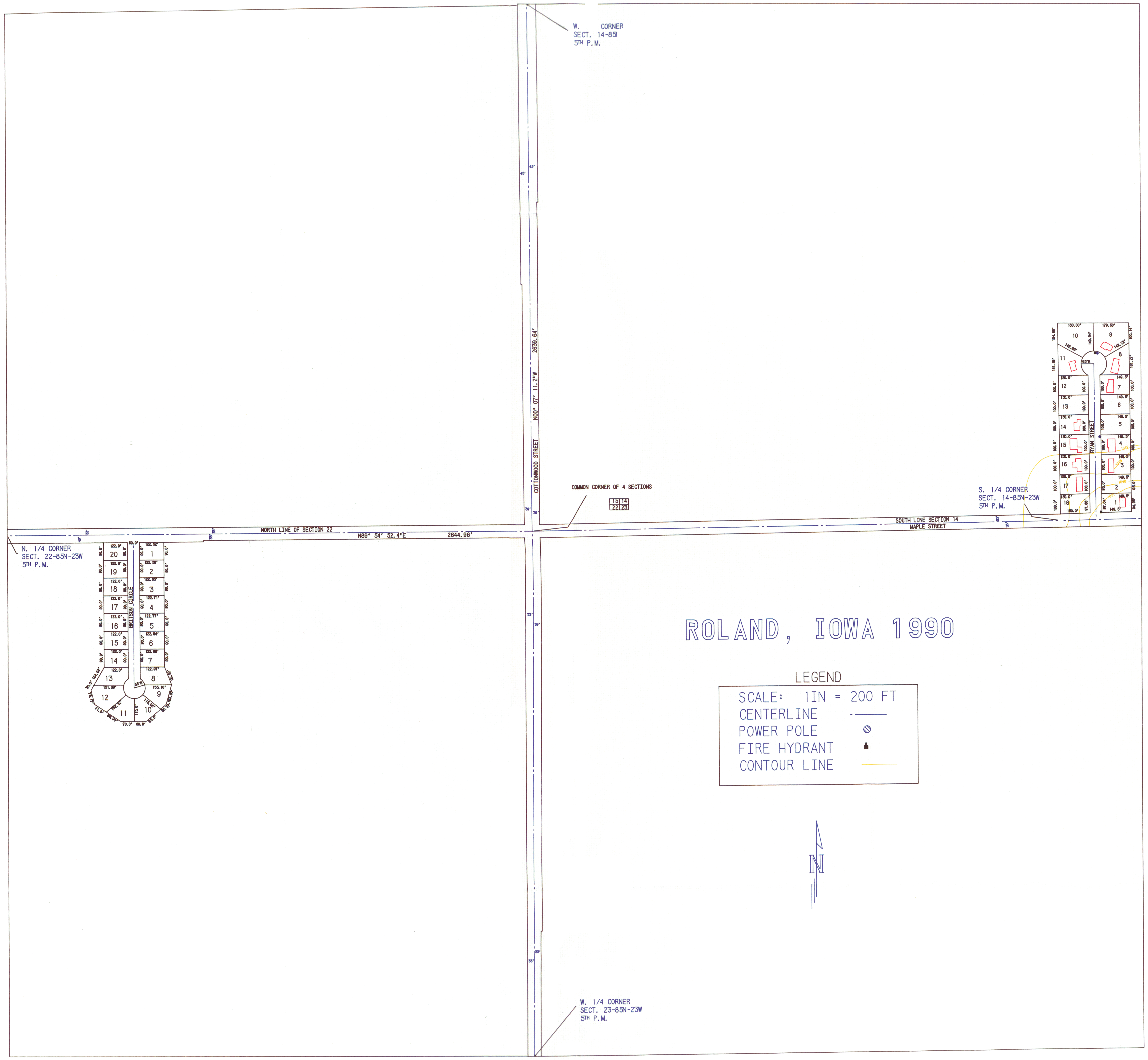
NORTHING: 3523413.037

EASTING: 4918693.864

APPENDIX I: LOT LINE Q-VIEW FOR BRITSON'S SECOND ADDITION

BEARING	DESCRIPTION
N.89DEG 54' 52"E.	NORTH/SOUTH LINE LOTS 18/19

APPENDIX J: MAP OF ROLAND, IOWA



W. CORNER
SECT. 14-85
5TH P.M.
COTTONWOOD STREET
N00° 07' 11.2"W
2639.64'
COMMON CORNER OF 4 SECTIONS
15 14
22 23
NORTH LINE OF SECTION 22
N89° 54' 52.4"E
2644.96'
N. 1/4 CORNER
SECT. 22-85N-23W
5TH P.M.
BRILLSON CIRCLE
S. 1/4 CORNER
SECT. 14-85N-23W
5TH P.M.
SOUTH LINE SECTION 14
MAPLE STREET
IOWA STREET
W. 1/4 CORNER
SECT. 23-85N-23W
5TH P.M.
ROLAND, IOWA 1990
LEGEND
SCALE: 1IN = 200 FT
CENTERLINE
POWER POLE
FIRE HYDRANT
CONTOUR LINE
N