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Developing a Geographic Information System for environmental studies on the Bear Creek watershed in Roland, Iowa

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Developing a Geographic Information System for environmental studies on the Bear Creek watershed in Roland, Iowa

by

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Signatures have been redacted for privacy

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CHAPTER 1

INTRODUCTION

Data acquistion for a Geographic Information System (GIS) can be done in several ways. Developing a GIS requires the establishment of a well-distributed ground control network and the preparation of a detailed base map. A base map upon which all the topographic features and contours are shown is an invaluable element in any GIS. Apart from serving as a check to the GIS, it serves as one of the main sources of data input. Developing the GIS for the study area therefore involved the initial task of preparing a base map at a scale of 1in to 50ft with a contour interval of 2ft. The objective of this study is to develop a GIS using Photogrammetry, Satellite surveying (the GPS) and ground survey methods of leveling, and radial survey by the total station.

The first step was to develop a GIS for a section of the Beer Creek watershed in Roland, Iowa. The Geographic Resources Analysis Support System (GRASS) GIS software was used. To establish the initial controls for the survey, points were set by the satellite positioning method of survey - the Global Positioning System (GPS). The extra controls needed were obtained by ground survey methods using the Geodimeter 424 total station survey instrument. In addition to the control survey by GPS which was done in the static GPS mode, a kinematic GPS method was used to determine the coordinate of the well locations for the coordinate comparison part of the study.

By using photogrammetric procedures, a base map was prepared which was digitized to form the input data for the GIS development.

The essence of the GIS was to be able to identify the relative locations of the wells and other features in the study area.

The second step was to analyze the coordinates obtained by the total station and the photogrammetric methods. The analysis was done in two ways. First determine whether there is any significant difference in the methods for positioning between the total station and the photogrammetric methods and second analyzing the economic implications associated with each method.

Although, there are a number of methods that could be adopted for GIS data acquisition, it is essential to select the best method to meet the standards and cost requirements of the project. A Comparison of the various methods of data acquisition by coordinates and cost effectiveness is done in Chapter 9.

Chapter 2 discusses the planning aspect of the study. Chapter 3 describes the Global positioning system. The principles and methods used are overviewed in this chapter. Chapter 4 explains the principles and work done with the Total station, and also introduces the theories of measurements and the methodologies. Chapter 5 deals with the leveling aspect of the thesis. Chapter 6 describes the coordinate systems used in the study. Chapter 7 describes the photogrammetric methods . The steps involved in preparing a base map are also explained and the aerotriangulation work is briefly discussed.

The GIS of the thesis is discussed in Chapter 8. Chapter 10 presents the conclusions of this study.

The coordinates of the wells in the three coordinate systems are in Appendix A. Appendix B contains the coordinates of all the grid points used to make the contours. These coordinates as explained later were obtained by photogrammetry. Hard copies of the maps and plots generated using the AutoCAD, Surfer and GRASS software are enclosed in Appendix C.

It should be pointed out that since (GRASS) software is more for Image processing than data management, its query capability is limited and therefore it could not be used extensively; however, a few queries could be done in the GRASS vector mode. The only use of the cell map was to display it.

CHAPTER 2

PLANNING AND RECONNAISSANCE

In order to achieve the objective of developing a GIS using photogrammetry, satellite surveying and ground survey methods of leveling and radial survey by total station, the Bear Creek watershed in Roland, Iowa was selected as the pilot study area. This GIS is for environmental studies. Roland is located at about 12 miles northeast of Ames.

The location of the wells were determined by ground survey methods, photogrammetry, and by the Global Positioning System (GPS). To obtain the absolute position of the well locations, ground points were set in the field. These points were also used to control both the ground survey and the photogrammetric methods.

The initial project phase which was a reconnaissance survey of the area was carried out in order to select suitable locations for the control points while taking into account the flying area and the intervisibility of points (see Figure 2.1).

In total, nine control points were selected and the locations of these points were marked with rebar rods and painted. Three witness marks were used for each of these points. Three points were selected on the south side of the study area, three spread across the center of the area, and the remaining three on the north side of the area. The relative locations of these points are shown in Figure 2.1.

Prior to flying the area, the control points and the wells to be

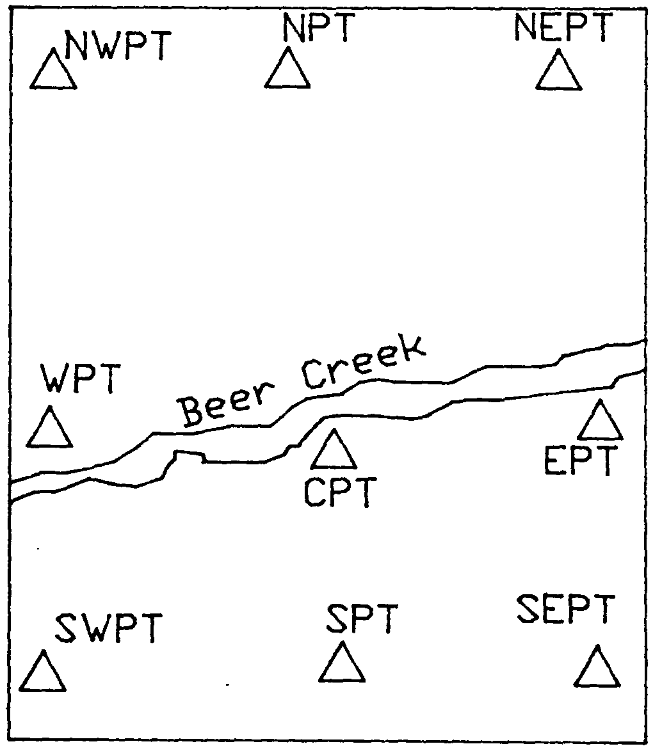

Figure 2.1: Relative positions of ground control points

studied were targeted with white plastic material. The purpose of targeting is to enable the targets to be clearly located on the photographs and the diapositives.

Because of the nature of the terrain and the time constraint, the ground survey methods of leveling and the total station were used for determining the elevations and coordinates of the control points, respectively. A two loop leveling was used for elevation determination. A relatively high point from which all the points could be seen was selected as a total station instrument station for the final coordinate determination of the control points and the wells. The method of radial survey was adopted.

Of the nine control points, the coordinates of two were determined by GPS. Only two were selected because static GPS observation is time consuming. To maximize the effective use of the GPS constellation and time, the GPS visibility program (Ashtech Inc.) was used to determine the best time and a satisfactory window to do the observation. Special emphasis was placed on the availability of satellites at a particular time slot, the time of day, and the PDOP value (see Chapter 3). A PDOP of three or less with an average of five satellites were considered good for observation throughout the entire GPS operations.

CHAPTER 3

GLOBAL POSITIONING SYSTEMS (GPS)

3.1 Description of GPS

The GPS is a worldwide all-weather spaced based positioning, navigation and timing system.

A full configuration of the system will consist of 21 satellites plus three spares making a total of 24 satellites. These will be maintained in a six evenly spaced circular orbits inclined at about 55° to the equator and at an altitude of about 20,200km with an orbiting period of 12 hours.

The system is so arranged that at full operation, a minimum of 4 satellites will be in view to users worldwide at any time of the day. Each satellite transmits two L-band frequencies; L1 and L2 (Figure 3.1).

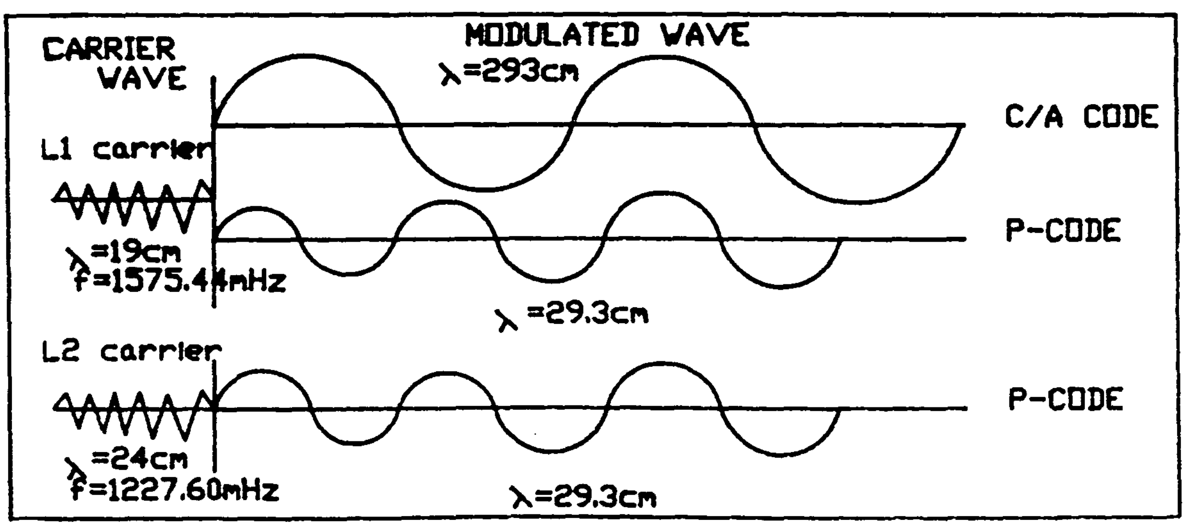

Figure 3.1: Signals broadcast by GPS satellites

Modulated onto these carrier waves is a code or navigation message containing satellite clock parameters, ephemeris, ionospheric corrections and other system information. The L1 signal is modulated with a precise or P-code and a coarse acquisition or C/A code while the L2 carriers are modulated with the P-code only.

The codes provide the time marks for determining satellite-receiver ranges and are therefore very essential for positioning.

The space satellites are controlled by a series of ground stations which track and monitor all satellites. From the tracking information, each satellite's status is monitored and precise orbit computations are carried out. Based on these computations, a daily orbit prediction is made and the information is uploaded daily onto each satellite to be transmitted with the navigation message as broadcast ephemeris.

3.2 Theory of GPS

As stated earlier, the orbital parameters of the satellites are constantly updated by control stations enabling the instantaneous position of the satellites to be calculated using the broadcast ephemeris to the receiver.

Suppose a satellite-receiver ranging is made from satellite S to receiver 1, then, the distance between the satellite and the receiver (usually called pseudo range due to errors associated with the measurement) may be defined as:

$$r_{1S} = [(X_1 - X_S)^2 + (Y_1 - Y_S)^2 + (Z_1 - Z_S)^2]^{1/2} + V(\Delta t_1 - \Delta t_S) + I_{1S} \qquad (3.1)$$

where X_S , Y_S , Z_S and Δt_S are the three dimensional coordinates and clock error of the satellite and X_1 , Y_1 , Z_1, Δt_1 are those of the station of receiver 1.

V is the velocity of propagation of the satellite's signal (V = $3x10^8$ m/s) and I_{1S} represents the ionospheric and tropospheric corrections. The satellite clock errors are generally known from the broadcast ephemeris. Therefore, to determine the X, Y, Z and Δt of the ground position, one needs to observe at least four satellites to enable the formation of four simultaneous equations with four unknowns.

However, various differencing techniques are employed to eliminate or minimize some of the errors associated with measurements.

For the same satellite S and a second receiver 2, equation (3.1) can be repeated as:

$$r_{2S} = [(X_2 - X_S)^2 + (Y_2 - Y_S)^2 + (Z_2 - Z_S)^2]^{1/2} + V(\Delta t_2 - \Delta t_S) + I_{2S} \qquad (3.2)$$

A single (or first) differencing between equations (3.1) and (3.2) will yield the following equation:

$$r_{2S} - r_{1S} =$$

$$[(X_2 - X_S)^2 + (Y_2 - Y_S)^2 + (Z_2 - Z_S)^2]^{1/2} - [(X_1 - X_S)^2 + (Y_1 - Y_S)^2 + (Z_1 - Z_S)^2]^{1/2}$$
$$+ V(\Delta t_1 - \Delta t_2) + (I_{2S} - I_{1S}) \qquad (3.3)$$

Obviously, a single difference eliminates the satellite clock errors. By similar reasoning, a second satellite and the two receiver stations can achieve a double difference. These eliminate the receiver clock errors, and also eliminate or minimize most of the systematic errors such as weather corrections inherent in the GPS observation. As seen from equation (3.3), a single difference will minimize the weather errors. However, the degree of minimization depends on how far the stations are separated from each other. Since weather conditions like most geographical conditions change with distance, the shorter the distance, the more accurate the differencing technique in minimizing the effects of the weather.

3.2.1 Geometrical Consideration of Satellite Positioning in GPS Survey

The effect of satellite configuration or geometry termed dilution of precision (DOP) is the ratio of the satellite positioning accuracy to that of measurement accuracy. Generally, DOP is a scalar representation of the contribution of the satellite configuration to the position accuracy. Ideally, the position of a point will be best determined if the satellites have a geometry similar to Figure 3.2.

P is a point whose X, Y, Z position is needed and S_1 , S_2 , S_3 and S4 are four observed satellites.

The most commonly used DOP is the PDOP which is the dilution of precision for three dimensional positioning. For a point with coordinates X ,Y , Z the PDOP is defined as

$$\text{PDOP} = \frac{\sqrt{\sigma_x^2 + \sigma^2{}_y + \sigma_z{}^2}}{\sigma_o}$$

where $\sigma_x, \sigma_y, \sigma_z, \sigma_o$ are the standard deviations in X, Y, Z and that of unit weight respectively.

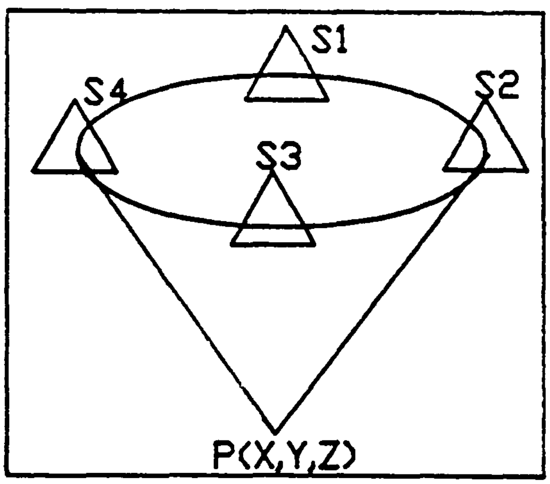

Figure 3.2: Ideal satellite geometry for GPS positioning

The standard deviation being measure of precision, it is evident that the lower the PDOP the better the positioning. A PDOP value of 3 is said to be good while anything above 5 may not yield satisfactory results.

3.3 **Observation Procedures**

Data acquisition by GPS can be done by one of several methods, namely; static, pseudo static, kinematic, semi-kinematic and pseudo kinematic. The methods adopted for the study were static and the semi-kinematic modes.

3.3.1 The Static Environment

The static method was used in the initial part of the survey to establish the coordinates of the southeast (SEPT) and the northwest (NWPT) points upon which all the other control points' coordinates were established. The entire GPS part of the survey was performed using the Ashtech LXII GPS receivers and antennas (Ashtech Inc.). The LXII is a single frequency (L1) unit of 12 channels (capable of tracking 12 satellites at a time).

One receiver was set at point TOWN (the control point located on the roof of Town Engineering building), one at the control point at Ames DOT and the third receiver was set at the point SEPT(in the field) whose coordinates were to be determined. The start of a GPS survey involves a few initial data entry into the receiver, such as the name of the station occupied, the minimum number of satellites to track, the minimum elevation of a satellite to track, and so on. After the required data entry was done, the receiver and the antenna configuration were left to collect data for 20 seconds per epoch for two hours. The system was then moved to the NWPT and the same routine was followed. A second session of observation was made between the two new points by setting one receiver at each of the points and collecting data for an average of 90 minutes (see Figure 3.3 for outline of survey).

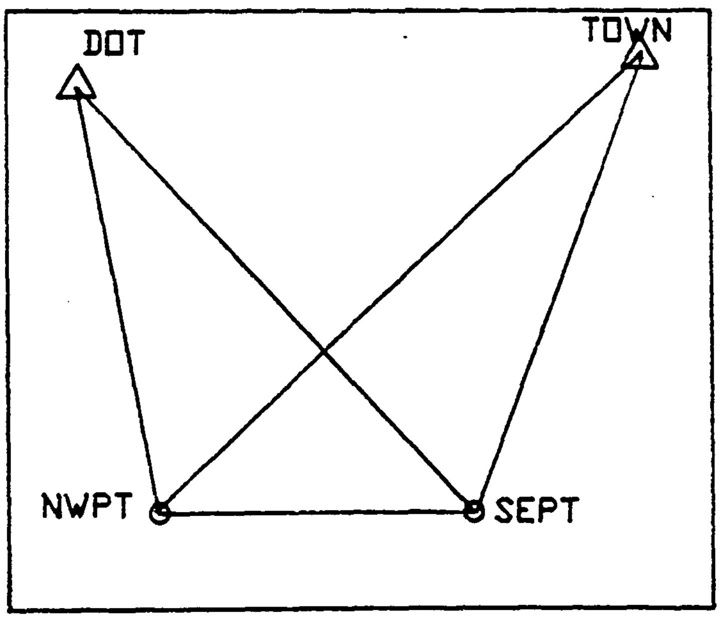

Figure 3.3: GPS static survey of Northwest and Southeast control points

3.3.2 The Semi-kinematic Environment (stop and go kinematic)

To achieve the best results, it is necessary to determine the integer cycle ambiguity before commencing any form of kinematic survey. The integer cycle ambiguity is the unknown number of whole carrier-phase cycles between each satellite-receiver pair at the commencement of the carrier phase tracking. The process of evaluating these ambiguities is sometimes referred to as "survey initialization". Survey initialization is done mainly by two techniques; antenna swap and known baseline methods. Antenna swap involves observing 6 to 8 epochs of data with receivers separated by a maximum distance of 10 meters. The antennas are then swapped for a similar time period , yielding the integer bias and the baseline vector.

For this project, the known baseline method was used since the coordinates of most of the wells had already been determined. Two wells were therefore selected and used to initialize the survey. Each receiver was set at one of these stations and about 10 minutes (60 epochs) of data were collected. With the coordinates of the two wells known and fixed, the integer bias could be determined and constrained during post processing.

After initialization, the rover receiver was moved to selected wells whose positions were to be compared with the total station method. An average of 5 minutes (30 epochs) data were collected at each station. A major difficulty in kinematic survey is moving the rover receiver and antenna connection from station to station. Care was taken to make sure the movement did not interfere with the communication link between any of the satellites and the receiver-antenna system.

A satellite can easily be dropped from lock if the operator inadvertently blocks the satellites' path or move the system around or under structures that can obstruct the satellite. Once a satellite looses lock during measurements, a cycle slip is said to have occurred which has an effect on the already determined ambiguity. Cycle slips are tolerated only when a minimum of four same satellites are still in continuous lock on the receiver. Otherwise kinematic survey requires that in the event of a cycle slip one goes back to the last position where a complete lock was attained and continue the survey from there. Maximum precautions were taken during this survey. Kinematic survey also requires that the first rover station (the other end of the base station) be revisited to end the survey.

To avoid any problems with persistent cycle slips, the mission planning program (Ashtech Inc.) was run to make sure that at least a minimum of five good satellites were available throughout the entire period of survey, with an average PDOP of three or lower. One interesting case is the effect of other electromagnetic signals on the satellite signals. In one particular case, an instrument equipped with electronic devices interfered with the satellite signals which did not permit observation to be made to the wells close to the instrument.

3.4 **Data Processing**

All the GPS data processing were done using the GPPS survey software (Ashtech Inc.). Using this software, the data were downloaded into the computer. Post processing involved running these data through a number of different programs in the software. The output of the processing are the geocentric and geodetic coordinates of the points.

These coordinates were then transformed to the Iowa State Plane Coordinate system via a coordinate transformation software (Coords83 by G.Willis Mahun, 1988).

CHAPTER 4

THE TOTAL STATION

The total station is a survey instrument that has the automatic data collection capability of angle and distance measurements. Thus, the total station may be seen as a theodolite and an electronic distance measurement (EDM) instrument fused together. The total station also has the capability of measuring elevation differences.

4.1 Principles of Measurements

Unlike the conventional theodolite, the total station measures angles by horizontal and vertical circular encoders whose output are stored in a data collector and entered into a built in microprocessor. The microprocessor also converts the measured slope distance into the corresponding horizontal distance using the measured vertical angle. The height difference between the instrument station and the reflector station is also computed.

4.1.1 Angle Measurement

Based on their mechanical structure, the angular measuring systems of conventional and electronic theodilites are quite similar. The fundamental element of both group of instruments is a graduated circle. Unlike the conventional theodolites, however, the reading microscope of the total station is a scanning system consisting of optical electronic elements. The angles are obtained in digital form and handled and stored by a computer which can be displayed digitally.

4.1.2 Distance Measurement

The distance measuring principle of the total station is not different from the conventional EDM instruments. The total station employs visible light as the carrier. The measuring set consist of the total station equipped with a transmitter and a receiver which is situated at one end of the line and a passive retrodirective prism reflector at the other end of the line. (Figure 4.1). A modulated light emission is transmitted to the reflector. The distance between the instrument and the reflector is computed by comparing the phases of the outgoing modulated waves with those received by the instrument after reflection from the target reflectors.

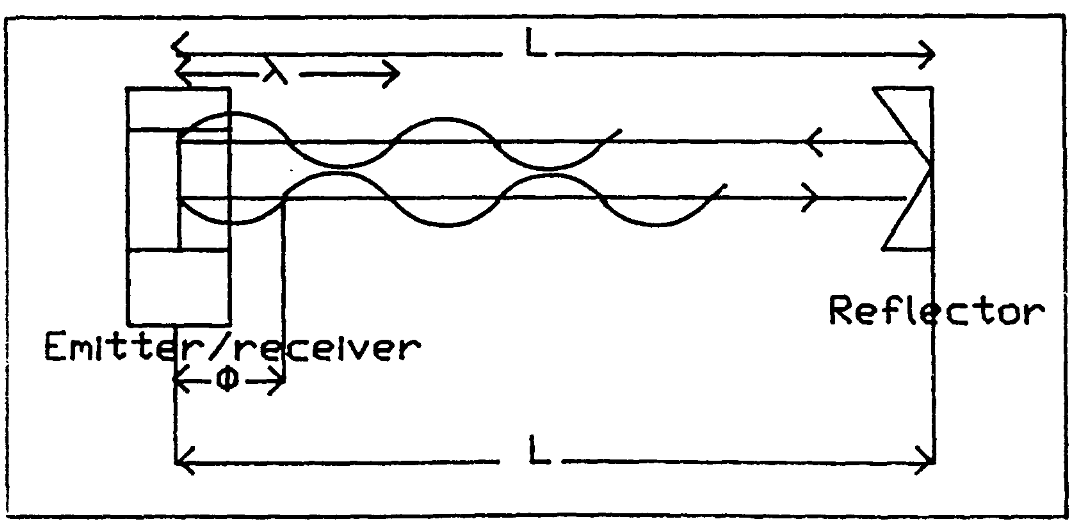

Figure 4.1: Principles of EDM Measurement

The total (double) distance covered is given by

$$2L = N\lambda + \Phi + k \quad (4.1)$$

$2L$ = double distance

N= number of complete wavelength obtained within the double distance.

λ= modulated wavelength

Φ= phase difference which is equivalent to a fraction of a whole wavelength of modulation.

The constant k is a correction factor for both the instrument and reflector combined.

Thus,

$$2L = N\lambda + \frac{\Phi}{2\pi}\lambda + k \quad (4.2)$$

The modulated wavelength may be expressed as

$$\lambda = \frac{C_0}{nf} \quad (4.3)$$

where c_0 is the velocity of electromagnetic wave in vacuum, n is the refractive index of the medium through which the wave passes and f is the wave frequency.

Thus, combining equations (4.2) and (4.3) we have;

$$L = \frac{NC_0}{2nf} + \frac{\Phi C_0}{4\pi nf} + k \quad (4.4)$$

Due to the varying effect of the atmospheric conditions, the velocity c_0 also changes. Hence atmospheric corrections are essential in electro-optical distance measurements. The total station allows the situation temperature, humidity, and pressure to be entered to calculate the necessary corrections and to apply them accordingly.

4.2 Field Procedure and Data Collection

Since many total station instruments have similar operation procedures, the procedure used in this project which is for the geodimeter 424 total station may be suitable for other total stations as well.

4.2.1 Operation Procedure

The Geodimeter 424 total station is designed for use with any standard tripod. The optical plummet attached to the instrument was used to center it over the instrument stations. The bulls eye bubble was then used to approximately level the instrument. Final leveling of the instrument was done using the digital bubbles (cursors). Once the instrument is leveled, it automatically goes to program zero which prompt the user to imput the temperature, pressure and the humidity. Weather conditions obtained from the weather station at the university were entered in the field. The height of the reflector and the instrument were also entered using the functions six and three, respectively.

4.2.2 Data Collection

The prime objective of using the total station was to get the X, Y, Z position of the wells and the remaining control points. The entire total station work was done by radial survey. A high point from which all the needed points could be sighted was selected as the instrument station. Since the total station has the capability of calculating the coordinates of the reflector position knowing the coordinates of the instrument station and a referenced azimuth, the immediate task was to determine the X, Y, Z position of the instrument station.

As described in chapter one, the coordinates of the SEPT and NWPT were known from the GPS survey. With the telescope sighting a reflector held at SEPT, the horizontal reference angle was set to zero and the X, Y, Z coordinates of the point were entered. The instrument was then turned to sight the NWPT and the X, Y, Z of the NWPT were entered. With this information the instrument computed the X, Y, Z position of the instrument station. Thus, using program three of the instrument, the name, code and the X, Y, Z positions of thirty-one wells and of the remaining seven control points were surveyed and stored in the instrument.

4.3 Data Processing

Data processing of a total station survey starts from downloading or transferring the data from the instrument or the data collector to the computer. Using the session codes and job numbers, the data were transferred to the external data collector and then to the computer using the geodimeter system 400 computer software. (Geotronics of North America Inc.) Since all the total station work was done by radial survey, the radial survey option of the program was used to process the data to determine the adjusted X, Y, Z positions of all the survey points.

CHAPTER 5

LEVELING

5.1 Principles of Leveling

The elevation of a point is the vertical distance above or below a reference datum. In surveying, the reference datum that is universally employed is the mean sea level (MSL). Leveling is the method used in determining the differences in elevation between points that are remote from each other.

5.2 Field Procedure

The mean sea level elevation of the SEPT and the NWPT were known by converting the GPS ellipsoidal heights to mean sea level heights. The prime objective of leveling therefore was to determine the elevations of the remaining seven selected control points. To determine these elevations, the method of differential leveling was adopted. Differential leveling is the method of determining the elevation differences of points using a surveyors level and a measuring rod.

Two leveling loops were designed for the entire area as shown in Figure 5.1.

The instruments used were; Wild automatic level, a leveling rod, and a foot plate which ensured a clearly defined rod position and also minimized the effect of sinking rod.

The procedure of observation was as follows; The level rod was set at the SEPT with the instrument set at a distance of about

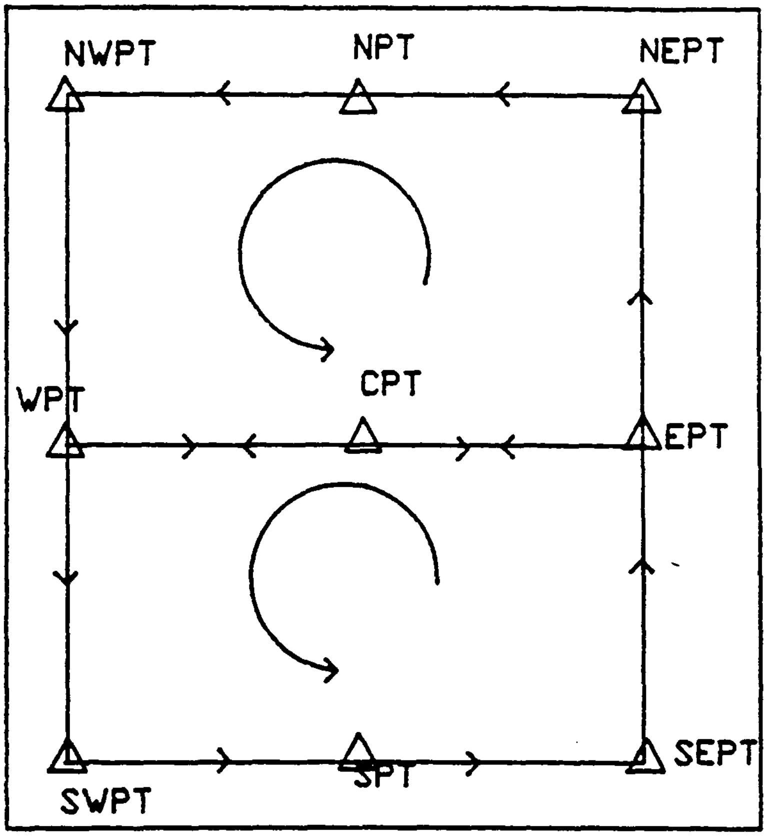

Figure 5.1: Leveling loops

30 meters from the point. A number of turning points were selected until the SEPT was revisited to end the first loop. The same procedure was followed in executing the second loop. Figure 5.1 depicts the loop arrangement of the survey.

5.3 Data Reduction

Consider Figure 5.2. Suppose the elevation of point A is E_A and the unknown elevation of B be E_B , then the following equations are valid:

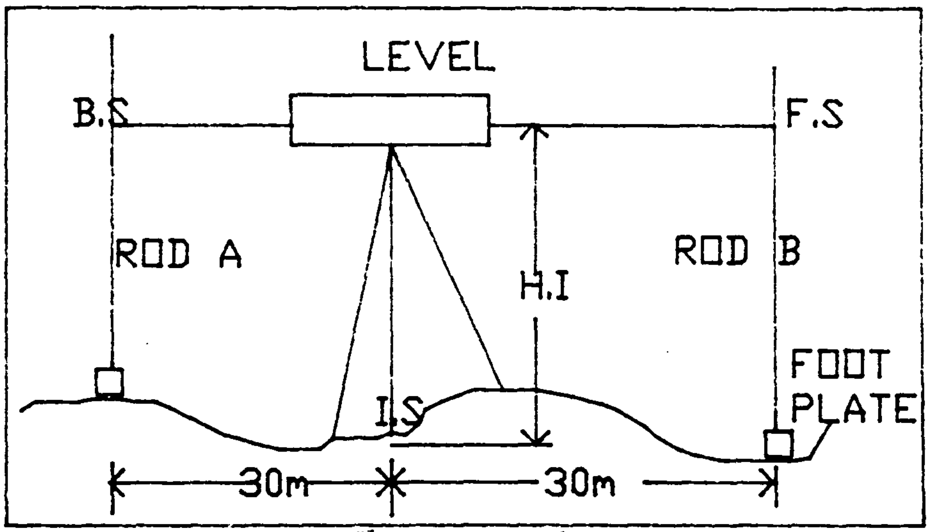

Figure 5.2: Typical instrument setup

$$HI = E_A + BS \tag{5.1}$$

$$E_B = HI - FS \tag{5.2}$$

The equal foresight and backsight distances are very essential to eliminate or minimize the effect of collimation error of the instrument.

In general the elevation of any new point is obtained by using equations (5.1) and (5.2).

The GPS elevations are elevations referenced to the GRS80 ellipsoid. Since the orthometric height of the points were needed, the GPS elevations were converted to MSL elevations by determining the geoid undulation of the area using a geoid undulation software. Figure 5.3 depicts the level surfaces.

h = The height above the reference ellipsoid (the ellipsoidal height).

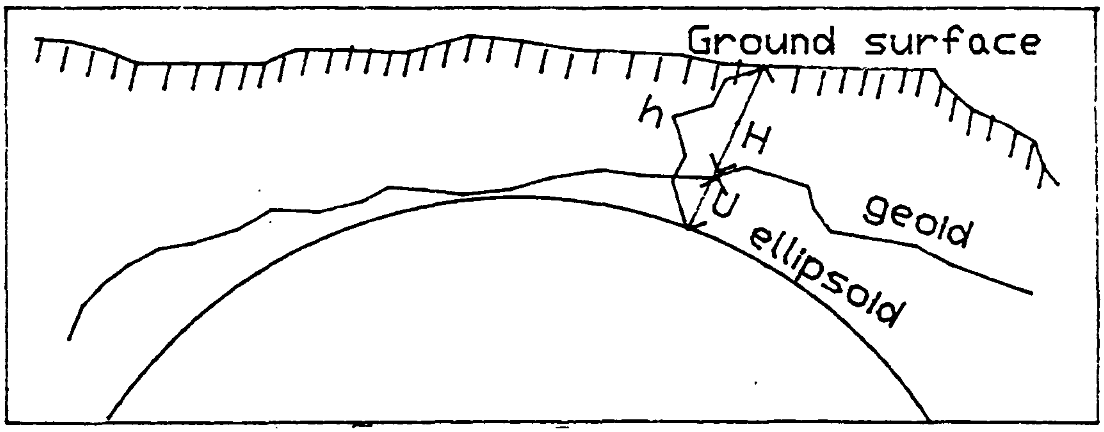

Figure 5.3: Level surfaces

H = The height above the geoid (the mean sea level elevation).

U = Geoid undulation (the separation between the geoid and the ellipsoid)

Thus,

$$h = H + U$$

After computing the geoid undulation of the SWPT, the corresponding MSL elevation was computed. Using the SEPT point as the starting elevation, the elevations of all the turning points, temporal points and those of the remaining seven control points were computed. The NWPT was used as a check point.

CHAPTER 6

COORDINATE SYSTEMS AND TRANSFORMATION

The main objective of surveying points on the Earth surface is to determine the positions of points relative to other points. The relationships between the multitude of points surveyed can only be established if the observations made on these points are used to establish coordinates of the points on one homogeneous coordinate platform. Such a coordinate platform may be referred to as a coordinate system.

In practice, many coordinate systems are employed depending on the circumstances involved. However, in this study we shall use three types of coordinate systems: the spherical coordinate system, the state plane coordinate system and the Universal Transverse Mercator (UTM) coordinate system.

6.1 The Spherical Coordinate System

Mathematically, the spheroid which is an ellipse rotated about its minor axis approximates closely to the shape of the earth. Being a close approximation to the earth and the fact that it can be specified in fairly simple terms mathematically, make the spheroid attractive as the reference surface unto which points on the earth surface are projected.

Points on the earth are located by a set of spherical coordinates; latitude (Φ) and longitude (λ). In the spherical coordinate system, the latitude of a point is referenced from the equator. The equator is then assumed as the line of zero latitude and points may be between 0 to 90 degrees south or north of the equator. On the other hand, the reference longitude referred to simply as the standard meridian is the meridian passing through Greenwich, England. The Greenwich meridian is assigned a value of zero and points may be 0 to 180 east or west of this meridian. See Figure 6.1.

Spherical coordinates were the coordinates obtained using the GPS survey method. These coordinates were then transformed to the

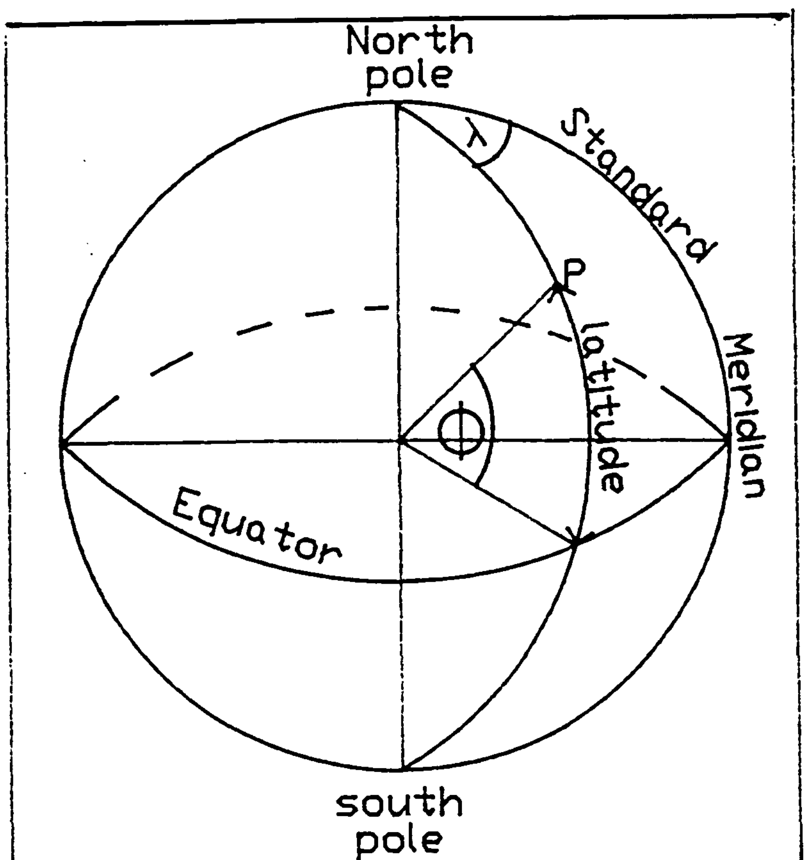

Figure 6.1: Spherical coordinate system

needed coordinate system using an appropriate transformation equations explained later in the chapter. A list of the spherical coordinates of all the points is given in Appendix A.

6.2 The State Plane Coordinate System

Unlike the spherical coordinates, state plane coordinates are in linear units of meters or feet and in Y and X rectangular system usually denoted by northings and eastings, respectively.

On a reasonably small area, surveying is considered to be on a plane in which the effect of the curvature of the earth is neglected. However, as the survey area becomes bigger and bigger, the validity of this "earth as a plane" assumption becomes weaker and hence dropped. It then becomes necessary that a suitable surface that is not a plane be used to project points on the earth's surface. In the state plane coordinate system, the geodetic positions of points on the spheroid are projected onto an imaginary solid surface. The state plane coordinate system employs two major surfaces for projection; the cone and the cylinder. The Lambert conformal conic projection uses the cone as the developable surface, while the Transverse Mercator uses the cylinder.

The type of projection used depends on the size and geographical orientation of the state. An essential feature of the state

plane coordinate system is to minimize the effect of projection distortion as much as possible. In terms of distortion, the Lambert conocal projection is more suitable for states with bigger stretch in the ease-west direction while the Tranverse Mercator projection is better for states with north-south stretch. A brief description of the Lambert conformal conical projection will be considered here because this projection is used in our study area.

6.3 The Lambert Conformal Projection

As stated earlier, the Lambert conformal projection employs the cone as the projection surface. In this type of projection, the cone intersects the spheroid along two parallels of latitudes called the standard parallels at one-sixth of the zone width from the north and south zone limits. All meridians are straight lines converging at the apex of the cone in the projection. In the projection, a meridian whose longitude is near the middle of the projection zone is selected as the central meridian. (see Figure 6.2)

The X and Y coordinates of a point P in the zone may be computed using the equations below.

$$\Delta\lambda = \lambda_{cm} - \lambda \tag{6.1}$$

where $\Delta\lambda$ is the difference in longitude between the central meridian (λ_{cm}) of the zone and the longitude (λ) of the point P whose position is to be projected.

As the longitudes converges at the apex of the cone, the convergence factor commonly referred to as convergence (θ) is calculated as:

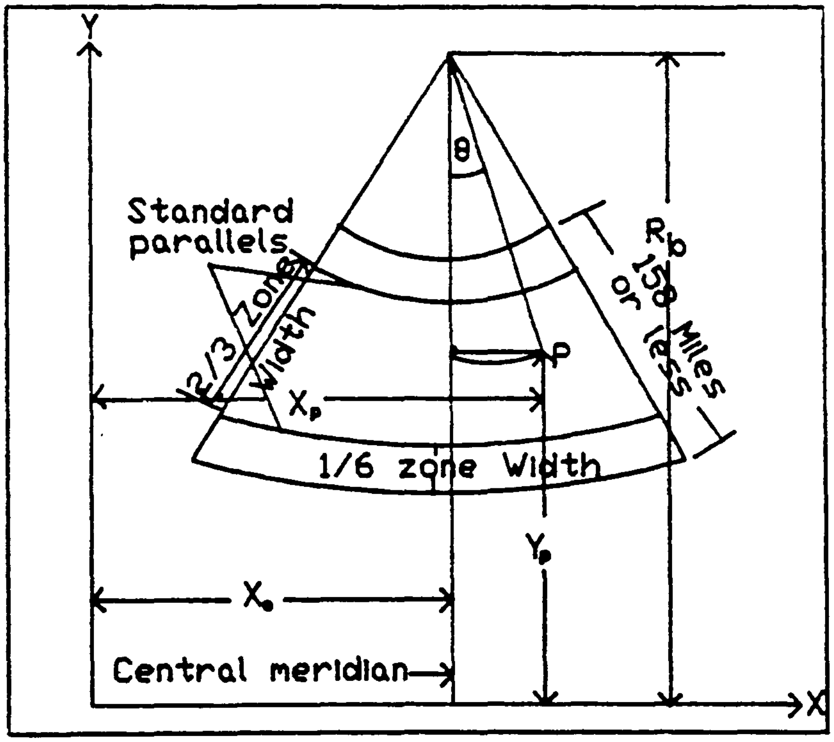

Figure 6.2: The Lambert conformal conic projection

$$\theta = l\Delta\lambda \tag{6.2}$$

where l is a constant of the projection.

The radius of curvature along the meridian R_1 is defined by

$$R_1 = \frac{a(1-e^2)}{(1-e^2\sin^2\phi)^{3/2}} \tag{6.3}$$

and the radius of curvature in the plane perpendicular to the meridian is given by

$$N = \frac{a}{(1 - e^2 \sin^2 \phi)^{1/2}} \tag{6.4}$$

The radius of the earth may be assumed to equal 20,906,000ft or may be computed from

$$R = \frac{R_1 N}{R_1 \sin^2 \alpha + N \cos^2 \alpha} \tag{6.5}$$

Where a and e are the semi major axis and the first eccentricity of the referenced ellipsoid, respectively. ϕ is the latitude of the point P and α is the geodetic azimuth between points.

The X coordinate of P may then be calculated as

$$X_p = X_o + R \sin \theta \tag{6.6}$$

and the Y coordinates as

$$Y_p = R_b - R \cos \theta \tag{6.7}$$

where X_o is a constant adopted to offset the central meridian from the y axis to avoid negative X coordinates. R_b is also a constant of the projection.

Iowa has two Lambert conformal projection zones. One covering the north with zone code 1401 and one covering the south with the code 1402. Roland is located in the northern part of Iowa and hence employs the northern projection.

The projection parameters of the north zone are;

Central meridian (λ_{cm}) = $93°30'w$

South standard parallel = $42°04'N$

North standard parallel = $42°16'N$

R_b = 1,000,000.000m

X_o = 1,500,000.000m

l = 0.677744602

6.4 The Universal Transverse Mercator (UTM) Projection

The UTM projection is based entirely on the transverse mercator projection by projecting the entire world onto a cylinder. The reference spheroid is the international earth ellipsoid of 1927. The UTM system divides the world into 60 zones of 6^{o} width. The origin of longitude of each zone is the standard meridian and the origin of latitude is the equator. The UTM coordinates are in meters only.

The UTM coordinate system was used in the GIS. Geodetic and state plane coordinates were transformed to the UTM system.

6.5 Transformation of Coordinates

As explained earlier, there are quite a number of coordinate systems in use for survey and other applications. The system selected depends on the application area and other factors. In most cases it becomes necessary to switch over from one system to another. This is accomplished by the process of coordinate transformation. Using the

"Coord83" transformation software, the geodetic and state plane coordinates were transformed to the UTM coordinate system for the development of the GIS.

CHAPTER 7

PHOTOGRAMMETRY AND MAP PREPARATION

The process of photogrammetry embraces the techniques and processes involved in taking photographs, measuring the photographs, and reducing the measurements to some usable form such as a map. Two aspects of photogrammetry were used in the study. One was using the photographs in making a map of the area and the second was measuring the photographs to determine the coordinates of the control points and well locations.

7.1 **Map Preparation**

The entire map of the study area was prepared using the Kelsh stereo plotter (Fig. 7.1).

The Kelsh plotter is an optical projection instrument used to draw maps using diapositives made from aerial photographs. The instrument is designed to allow for two overlapping photographs to be viewed at a time.

7.1.1 Mapping Procedure

The techniques involved in using the Kelsh plotter and the diapositives in making a map may be divided into two parts; preparing the model and plotting the model.

7.1.2 Model Preparation

Model preparation involves the process of loading the diapositives unto the instrument's photo plate carriers, orienting the model, and scaling and leveling the model. These processes may be classified into three main groups as inner orientation, relative orientation and absolute orientation.

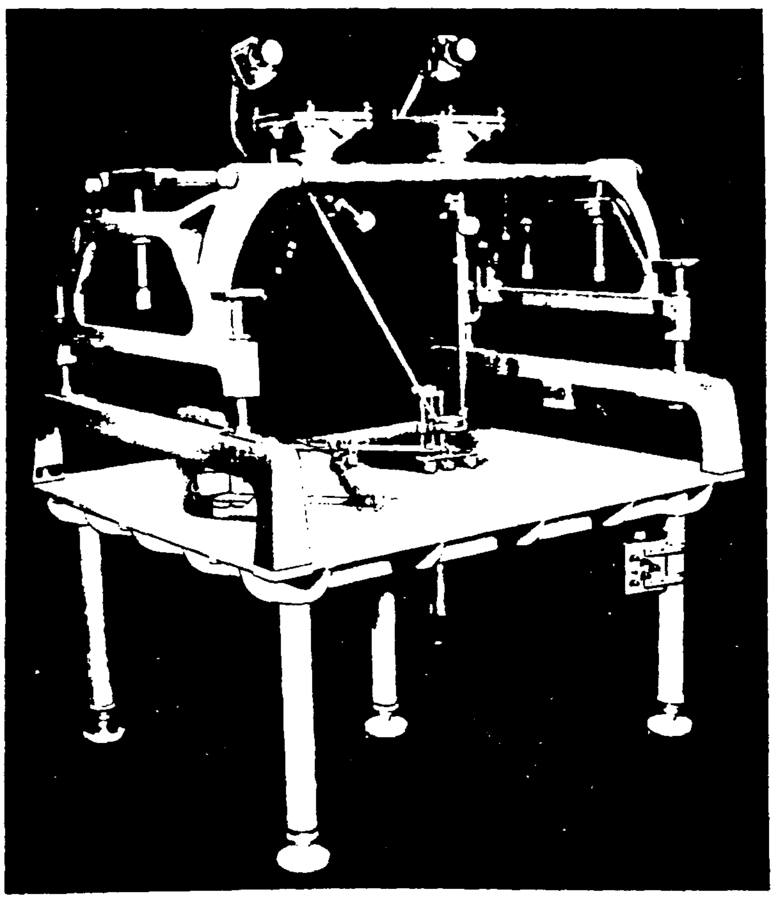

Figure 7.1: The Kelsh plotter

7.1.3 Inner Orientation

Inner orientation defines the bundle of rays that passes the perspective center of the lens to the object space during aerial photography. Inner orientation was performed by putting the left diapositive to the left plate carrier of the instrument and aligning the fiducial marks of the diapositives with those of the plate carrier. The same procedure was repeated for the right photo carrier.

7.1.4 Relative Orientation

Two rays emerging from the same point located on both diapositives are supposed to converge and define the model point. This is usually not the case and the situation that arises is schematically represented by Figure 7.2.

The corresponding rays from the photo points a_1 and a_2 fail to intersect and create the model point after reaching the tracing table of the Kelsh plotter. The mismatch is resolved into two components called x and y parallaxes denoted by P_X and P_Y, respectively.

The x parallax which is a function of elevation can easily be eliminated by raising or lowering the tracing table or the projection table. Removing the y parallax in the model is accomplished by the process of relative orientation.

Like most plotting instruments, relative orientation on the Kelsh plotter is an iterative process. Six points on each model were selected fot this procedure. The relative locations of the six points

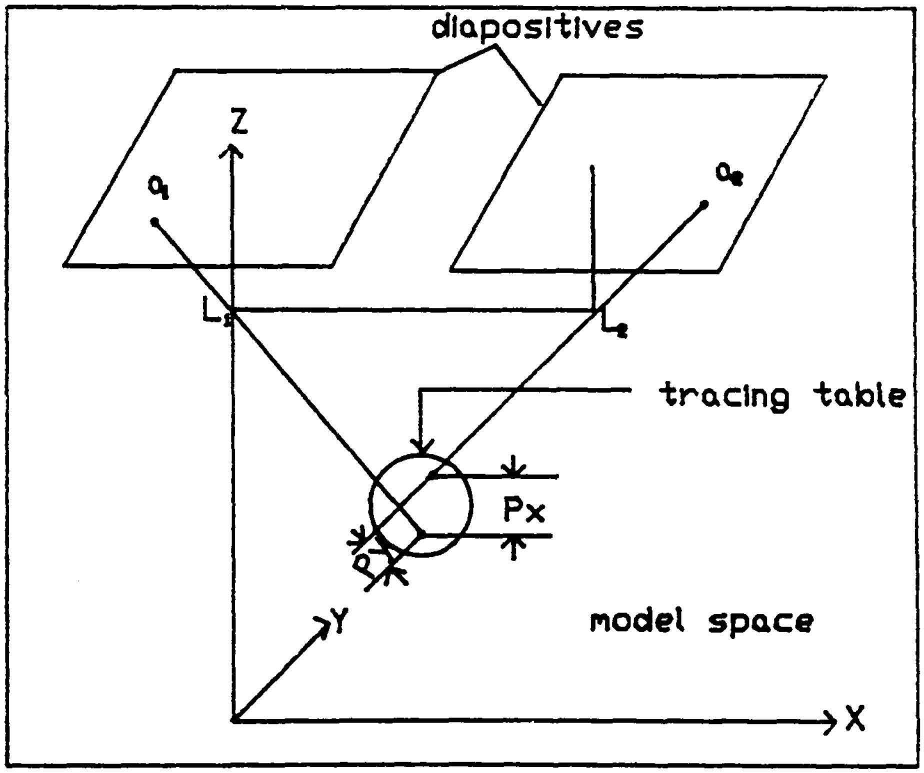

Figure 7.2: x and y parallaxes in a photo model

selected for each model is depicted in Figure 7.3.

The image of the model was projected to the tracing table for viewing. In all, there were four photographs of three models for the study area. For each model, six distinct points that appear in the area of overlap (see Figure 7.3) were selected for relative orientation.

The double projection method of relative orientation was used. The whole process of relative orientation with the Kelsh plotter involves a systematic and iterative corrections of y-parallaxes at five of the six selected points using the orientation elements of

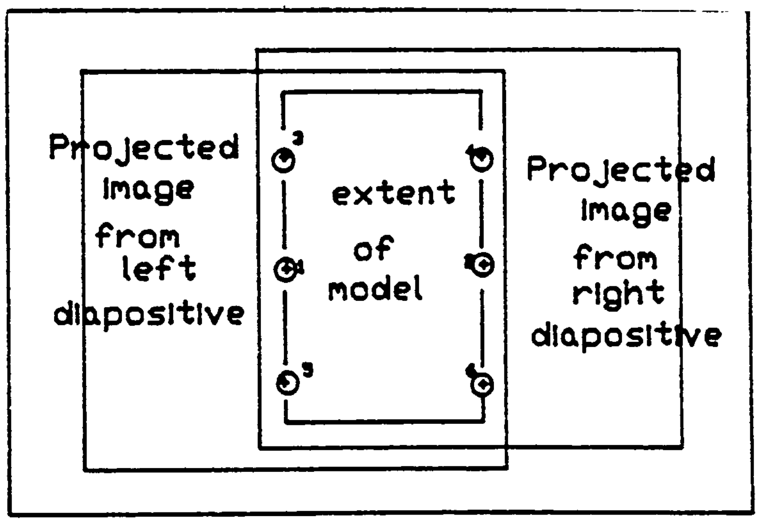

Figure 7.3: Relative orientation points in a photo model

Kapper1 (κ_1), Kapper2 (κ_2), Phi1(Φ_1), Phi2 (Φ_2) and Omega1 (ω_1).

The arrangement below indicates how the points were visited and the corresponding orientation elements used.

point	corrector
2	κ_1
1	κ_2
4	Φ_1
3	Φ_2
5	ω_1
6	check point

About five iterations were done to create a parallax-free model.

7.1.5 Absolute Orientation

Absolute orientation was done in two parts of leveling and scaling after relative orientation.

7.1.5.1 Scaling

Scaling is done to bring the scale of a photo model to that of the map scale. Two of the control points already plotted on the gridded map sheet at a scale of 1in to 50ft (the scale of the map) were selected. The plotting paper was oriented so that as the floating mark on the tracing table was on a model point, the plotting pencil was on a corresponding point on the map sheet. This process also involved series of iteration procedures by changing the bx of the instrument to reduce or increase the scale to the desired scale. The paper was then firmly taped to the drawing table. The next stage was leveling.

7.1.5.2 Leveling

Leveling is done to level the model so that relative height changes in the model will correspond to those on the ground. Three control points (plotted on the map sheet) were used for leveling each model: two points in the general y direction and the third in the x direction. Using the z counter of the instrument, the scale and the computed elevation differences on the ground, the model was leveled by iteratively raising and lowering the instrument using the attached screws.

7.1.6 Plotting

Once the model was fully oriented, plotting was not difficult. The outline of the features in the model were traced using the pencil of the tracing table. Roads, buildings and trees were drawn first. Before drawing these features, it is very essential that the x parallaxes that exist in the model be removed. Following the floating point through elevation changes was very difficult and hence it could not be used in plotting the two foot contours needed.

To get the contour lines therefore, elevation readings were made at every grid intersection of the plotting paper. These readings which were obtained using the Z-counter of the tracing table were transformed to the elevations of the corresponding points and the X, Y coordinates of the points were entered as input data in the Surfer program (Golden Software Inc.). By repeating the whole process for all the three models, the whole area was plotted and a complete X, Y and elevations of all the points observed were obtained and entered in the Surfer program to generate the contours. A complete contour map of the entire area was obtained from the Surfer program and was carefully transferred to the plotting paper. The contour interval used was 2 feet and the map was made at a scale of 1 inch to 50 feet.

7.2 **Aerotriangulation**

Aerotriangulation is the process of extending controls based on the coordinates of measurements of a limited number of points joined together numerically.

The spatial orientation and position of the photographs are determined by the method of space resection. Space resection is the process in which the spatial position and orientation of a photograph is determined based on the images of ground control points appearing on the photograph.

Space resection employs the collinearity condition (that the image point, object point, and the perspective center of the photograph all lie in a straight line) to determine the refined photo coordinates (see Figure. 7.5).

From Figure. 7.5 the following equations can be written:

$$\begin{bmatrix} X_P - X_L \\ Y_P - Y_L \\ Z_P - Z_L \end{bmatrix} = \lambda_P . R \begin{bmatrix} x_p \\ y_p \\ z_p \end{bmatrix} \tag{7.1}$$

where λ_p is a scale change and R is a combined rotation matrix defined as:

$$R = R_z . R_y . R_x \tag{7.2}$$

The following collinearity equations may be written for point p.

$$x_p = -f\left[\frac{m_{11}(X_p - X_L) + m_{12}(Y_P - Y_L) + m_{13}(Z_P - Z_L)}{m_{31}(X_p - X_L) + m_{32}(Y_P - Y_L) + m_{33}(Z_P - Z_L)}\right]$$

(7.3)

$$y_p = -f\left[\frac{m_{21}(X_p - X_L) + m_{22}(Y_P - Y_L) + m_{23}(Z_P - Z_L)}{m_{31}(X_p - X_L) + m_{32}(Y_P - Y_L) + m_{33}(Z_P - Z_L)}\right]$$

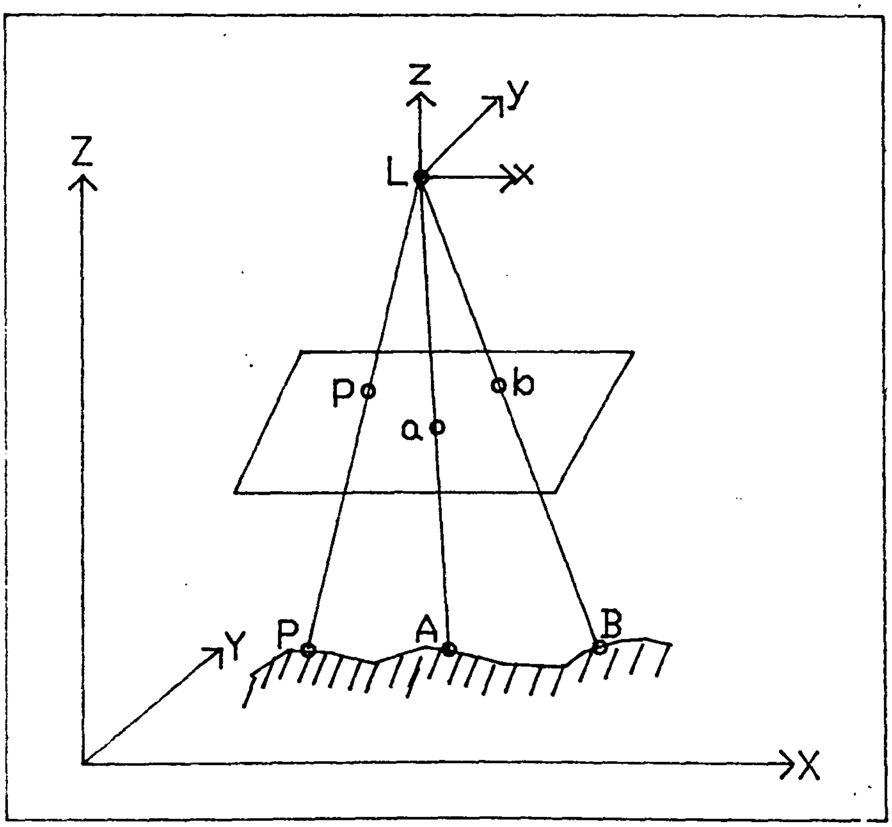

Figure 7.5: Collinearity

where x_p, y_p are the refined photo coordinates of the control point P; f is focal length of the aerial camera, X_P, Y_P and Z_P are the object space coordinate of the control point P and X_L, Y_L, and Z_L are the coordinates of the perspective center and $m_{11}.......m_{33}$ are the elements of the orthogonal matrix $M = R^{-1}$.

7.3 **Observation Procedure**

The Wild stereo comparator model stk-1112 was used as the observing instrument. The initial model preparation involved in using this instrument is not different from conventional stereo plotters as the Kelsh plotter explained earlier. After the fiducial marks of the photos are aligned with those of the plate holders and mounted onto the instrument, observation proceeds. To observe a point, the left floating mark is put on the particular point on the left diapositives and the right floating mark on the corresponding point on the right diapositives. For proper viewing, a 20x magnification was always set and the correct light intensity set to suit the eye.

A model observation starts with the observation of the eight fiducial marks. These points were observed and run through a program call "SAT9" to check on the accuracy of the measurements. A 20micron maximum error was allowed throughout the entire measurements. After observing to the fiducial marks and running the "SAT9" program, the observation process then continued with the well and control points location. In all, there were four photographs of three models. The data collected in observation were the x and y photo coordinates of the points and the p_x and p_y values. These values were entered into the computer as the input file for the SAT9 program.

7.4 **Data Processing**

Processing the data to get the final coordinates of the observed points using the PAL/MAPP/ALBANY computer program(ERIO Technologies) involved a number of steps. A relative orientation is first performed on the observed points by running a program called RO. The input file for this program is a specially formatted output file from the SAT9 program. The relative orientation program was run on each model and the output files obtained were joined together to form the strip.

In most cases, it became very essential to go back to the instrument and re-observe some of the models. These problems generally arose when there were bigger residuals in some of the observed points. However, in cases where no evidence indicated a measurement error, some of the observed points were considered as bad points and had to be removed from the file.

Now, with an input file made for the whole strip, the strip option of the PAL/MAPP/ALBANY program was run. To run this program, a number of support files had to be created. These include a control file which basically contains the X, Y, Z ground coordinates of the nine control points, a camera data file which contains the parameters of the camera used for photography, like the focal length. The output of the strip program is the X, Y, and Z ground coordinates of the points observed in the models. The ALBANY option of the program was also run.

CHAPTER 8

DEVELOPING THE GIS

Depending on the area of application, a geographic information system may be defined differently. Without any loss of generality, a GIS may be considered as any system capable of giving information on geographical structures and conditions. A GIS may not necessarily be seen as a computer assisted system only. For instance, a base map on a wall showing the distance between two cities, the difference in elevation between two points, the direction of flow and location of a river and a bus schedule in a bus station showing the time and location of a bus may be viewed as a geographic information system.

However, the recent interest and advancement in computer technology has revolutionized the entire idea of a GIS and hence the new generation of GIS concepts. Now, a GIS may be seen as "a system of hardware, software and procedure defined to support the capture, management, manipulation, analysis, modeling and the display of spatially referenced data for solving complex planning and management problems" (16).

8.1 Data Formats

Geographic data may be stored in two basic formats; raster and vector. In a raster data format, the area of study is divided into uniform units (cells) whose characteristics and attributes are then defined. In this structure, every cell represents a portion of the area (defined by the user) such as a square. Each cell is given a numerical value that corresponds to the feature, attributes or characteristics that is located at or describes the site. On the other hand, vector systems define an object and then identify its characteristics and attributes, including location.

Because a cell-based system modelling tool has different strength than a vector tool, the two systems should be seen as complimentary rather than competitive.

The era of choosing between raster or vector-based GIS softwares may soon be over as various software developers have began integrating the two systems together as in ARC/INFO 6.0 from Environmental Systems Research Institute (ESRI).

8.2 The GRASS GIS Software

GRASS (Geographic Resources Analysis Support System) is a public domain image processing and GIS system developed by the U.S Corps of Engineers Construction Research Laboratory in Champaign, Illinois, USA. GRASS was initially developed for military applications but now, it is used by many groups for other applications as well.

Both raster and vector data are accommodated (to a different extent) in GRASS. Vector data in GRASS are used for digitizing and graphic overlay of data. All data analysis in GRASS however are done on raster data.

GRASS is more of raster than a vector system and hence it is sometimes referred to as a raster-based GIS.

Included in its vector capabilities is the conversion programs that translate vector data to raster formats and vice-versa.

Like most GIS software, GRASS is capable of data digitizing, data read-in and read-out conversion, image processing, data analysis and data presentation.

8.3 Developing the GIS with GRASS

In this project, the main objective of developing the GIS was to be able to identify the location of the wells.

One major feature of developing a GIS is the source of data and method of data entry. In this case the source of data was the map prepared in Chapter 7. The AutoCAD DXF (Digital Exchange Format) file of the digitized map was used as the input data. Although, GRASS is capable of converting a DXF file formats to GRASS vector formats, there seemed to be a problem with the direct AutoCAD-GRASS transfer and hence the DXF file was first transferred to DLG (Digital Line Graph) through Arc/Info and then from Arc/Info to GRASS. With the data in the GRASS vector format, the next step was to create a mapset and location for the file from which the data could be accessed and processed. The location "rob" and mapset "PROJECTS" were then created for the file.

Since the map was digitized using the polyline option in AutoCAD, all the multitude of lines and areas had attributes attached to them totaling 775. To attach the real attributes like roads, contours and streams to the corresponding features, the AutoCAD attributes had to be detached from the features.

Almost all the work done on GRASS was done using the GRASS version 3.0 on the UNIX system.

The public domain aspect of GRASS made it very easy to work on the system, as one could easily log on any Vincent station as long as it had a color monitor for map display. To start GRASS on a vincent work station, the command "add grass" is typed at the Vincent prompt and a menu showing the various versions that the system supports is displayed. In this case the selected option was GRASS3.

After selecting the mapset and location to work with, a color monitor for map display was also selected. The color monitors supported by most of the Vincent machines are about 12 of varying dimensions. Since some of the monitors were not always available, the monitor that was usually used was the X2 (Xwindow display 900 x 750).

As said earlier, the development of the GIS started with the cleaning of the data. This was done using the GRASS program "DIGIT". Under the "DIGIT" program are a number of sub-programs that were used in the cleaning process namely; Edit, Label, Customize, Toolbox, Window, Help, and Quit.

A brief description of how the following sub programs were used is explained below.

8.3.1 Edit

Although a lot of editing was done on AutoCAd after digitizing, there still remained some editing after the transfer. The Edit option has a number of sub menus which were used here. The first task was to snap all the nodes in the lines to create a complete line for every unique contour or road. This was done using the menu item "s (snap nodes)". With this command the broken lines were connected. Also, there was one instance that a line needed be connected to another line. The gap that existed might have been created by the transfer process. The option "t (re-type a line)" was used to connect these lines. There are a lot of other options under this menu which were not used because they were not needed such as "R (remove block of lines)", "m (move a point)", "b (break a line)" etc.

8.3.2 Labeling

Labeling is the process of assigning category values to digitized objects. The category values which were automatically attached to the features during digitizing had to be replaced with the proper values. To do this, all the labels had to be unlabeled using the "Un-label" command in the label menu. There are two unlabel commands in this menu;

"A(Un-label areas)" and "L (Un-label lines)". The L option was used to unlabel all the lines and the A option was used to unlabel all the area features such as the wells which were represented as circles and the buildings which were represented by polygons. After removing all the 775 line and area labels, the features were then relabeled using the "l (label lines)" and "a (label areas)". To facilitate easy display, all the contours were given a category value equal to their contour value. For example the category value 1051 was given to the contour with elevation 1051 feet. All the wells were given a category values in the same way as they were numbered during the survey. The few buildings were given a category value of 12 and 11 for trees.

In the digit mode, one can easily identify the location of a particular contour or the locations of the wells by using the sub menu "s (show lines of category #)" and "d (display areas of category #)".

8.3.3 Window

Once GRASS is initiated, a window is automatically set which can be changed any time using the window option. A few queries can be done on vector maps under the window options. Most of the queries were done in "DIGIT". By using the command "i (show lines)", all the lines could be shown. In fact one could display only the lines in the map by temporarily disabling all the other features and their attributes. Similarly, only the areas, area labels, line labels, nodes could be displayed using various commands in the window menu. Most of these options were used to check whether all the lines and areas were labeled as desired.

8.3.4 Customize

The customize option has a lot of submenus but the two most commonly used during this process were the "D (enter display option menu)" and "C (enter color option menu)". These menus were used to set different colors for different features, disable features which need not be displayed etc. Under the display option an off and on toggle was used to disable or display features.

8.3.5 Vector to Cell

Converting the vector map to a grass raster map was the final step. This was done using the command "vect.to.cell". Before this conversion, the vector map was run through the support files which created some other files needed in the conversion. To convert a vector map to a cell map, all the features need to have a category number attached to it. Features without category numbers cannot be converted and appear as blank spaces when displayed. The category and history files of the newly created cell map can be updated by running the support program. The cell map was only used to display.

CHAPTER 9

ANALYSIS OF RESULTS

Analysis of results were in two parts. First, was to compare the coordinates obtained by each of the three methods and analyze them to determine whether there are significant differences in their use for GIS. And second, was to determine the economic implications of the methods used in data acquisition, vis, the Total station, GPS, and Photogrammetry

9.1 Coordinates Comparison

A comparison among the common points surveyed with the three methods is considered here.

Table 9.1 shows the coordinates of 20 of the points surveyed with the total station and Table 9.2 shows coordinates obtained by photogrammetric aerotriangulation.

To compare the coordinates obtained by these two methods, the difference between coordinates from the total station and photogrammetry were taken as depicted in Table 9.3. Here the coordinates from photogrammetry were subtracted from those from total station. As in the table, the difference between the z coordinates are smaller than those of X and Y. The maximun difference were 0.254m, 0.522m and 0.238m and the minimum were 0.009m, 0.003m and 0.006m for Y, X and Z respectively. 0.104m, 0.125m, and 0.089 are the standard deviations in difference in X,.Y and Z respectively.

Table 9.1: Coordinates obtained from the Total station survey (m)

WELL #	Y	X	Z
21	1076827.82	150090.87	316.768
22	1076827.95	1500828.88	319.204
23	1076704.07	1500716.01	316.266
24	1076659.17	1500813.75	316.789
27	1076630.23	1500789.34	316.880
28	1076632.64	1500781.62	316.849
29	1076614.89	1500756.78	316.988
31	1076629.12	1500752.47	316.678
32	1076634.93	1500750.75	316.555
33	1076656.41	1500729.50	316.059
34	1076657.34	1500729.38	316.017
35	1076621.92	1500726.87	316.684
37	1076614.29	1500700.67	316.547
39	1076622.29	1500689.13	316.314
40	1076597.76	1500677.86	316.957
44	1076711.76	1500853.67	317.076
45	1076755.08	1500843.46	316.589
46	1076779.80	1500851.46	316.647
48	1076788.90	1500961.50	318.623
49	1076637.80	1501045.27	320.894

Table 9.2: Coordinates obtained from Photogrammetric (m)

WELL #	Y	X	Z
21	1076828.03	1500940.90	316.706
22	1076828.10	1500828.88	319.159
23	1076704.05	1500716.14	316.235
24	1076659.23	1500813.91	316.894
27	1076630.28	1500789.61	316.908
28	1076632.65	1500781.76	316.931
29	1076614.77	1500757.30	317.019
31	1076629.13	1500752.71	316.725
32	1076634.96	1500750.98	316.689
33	1076656.38	1500729.63	315.943
34	1076657.30	1500729.59	316.032
35	1076621.86	1500727.08	316.667
37	1076614.23	1500700.92	316.566
39	1076622.27	1500689.36	316.305
40	1076597.70	1500678.03	316.962
44	1076711.84	1500853.82	317.070
45	1076755.17	1500843.55	316.553
46	1076779.94	1500851.51	316.481
48	1076789.10	1500961.49	318.861
49	1076638.06	1501045.54	320.999

Table 9.3: Difference in X, Y, and Z coordinates between Photogrammetry and total station (m)

WELL #	ΔY	ΔX	ΔZ
21	0.214	0.033	-0.062
22	0.154	-0.003	-0.045
23	-0.022	0.134	-0.031
24	0.060	0.167	0.105
27	0.045	0.267	0.028
28	0.006	0.146	0.082
29	-0.120	0.522	0.031
31	0.009	0.235	0.047
32	0.036	0.230	0.134
33	-0.025	0.133	-0.116
34	-0.038	0.212	0.015
35	-0.055	0.216	-0.017
37	-0.064	0.254	0.019
39	-0.023	0.230	-0.009
40	-0.059	0.171	0.005
44	0.076	0.152	-0.006
45	0.095	0.087	-0.036
46	0.146	0.050	-0.166
48	0.202	-0.010	0.238
49	0.254	0.267	0.105
Sigmas:	±0.104	±0.125	±0.089

In addition to this comparison, the difference between the total station and the semi-kinematic survey coordinates were compared as shown in Table 9.4. This table reveals that the two coordinates do not significantly differ and that either of the two methods may be suitable for data acquisition. The big difference of about 3m at well 40 may be due to the sharp change of PDOP from 2 to 10 when station 40 was occupied for the first few minutes.

Table 9.4: Difference in coordinates between the total station and semi-kinematic survey (m)

WELL #	ΔY	ΔX	ΔZ
36	-0.006	0.046	0.056
37	0.037	0.072	0.084
38	0.085	0.080	0.075
40	2.863	0.009	-0.050
41	0.199	-0.087	-0.085
42	0.310	-0.007	-0.124
Sigmas:	±0.13	±0.06	±0.09

9.2 Statistical Analysis

To analyze the relationship between the coordinates obtained between the two methods, the standard deviations were computed and hypothesis testing was done.

The equations used in computing the coordinates of the points are summarized below.

$$\begin{aligned} X &= X_o + sSin\alpha \\ Y &= Y_o + sCos\alpha \\ Z &= Z_o + \Delta Z \end{aligned} \qquad (9.1)$$

where (X_o, Y_o, Z_o) are the coordinates of the instrument station from which points were radiated. s is the distance between the instrument station and the target while α is the azimuth between stations. ΔZ is the difference in elevation between the two stations.

By the method of propagation of errors using Taylor's series approximation and also assuming that the two methods are completely independent, the following equations are derived for the variances of the computed coordinates.

For X,

$$X = X_o + sSin\alpha$$

$$\sigma_x^{\ 2} = \left[\frac{\delta x}{\delta x_o}\right]^2 \sigma_{x_o}^{\ 2} + \left[\frac{\delta x}{\delta \alpha}\right]^2 \sigma_\alpha^{\ 2} + \left[\frac{\delta x}{\delta s}\right]^2 \sigma_s^{\ 2}$$

$$= \sigma_{x_o}^{\ 2} + (sCos\alpha)^2 (\frac{\sigma_\alpha}{\rho}) + (Sin\alpha)^2 \sigma_s^{\ 2}$$

where ρ=206265 (a factor that converts seconds of arc to radians).

By similar reasoning the variances of Y and Z may be obtained as;

$$\sigma_y^{\ 2} = \sigma_{y_o}^{\ 2} + (sSin\alpha)^2 (\frac{\sigma_\alpha}{\rho}) + (Cos\alpha)^2 \sigma_s^{\ 2} \quad \text{and}$$

$$\sigma_z^{\ 2} = \sigma_{z_o}^{\ 2} + \sigma_{\Delta z}^{\ 2}$$

By using these equations, the standard deviations were computed as ±2cm, ±4cm and ±4cm for X, Y and Z respectively.

9.2.1 Hypothesis Testing

The whole idea of this statistical analysis was to determine if the coordinates obtained by the two methods are significantly different. Thus a hypothesis testing was performed to ascertain this fact.

The sample is assumed to come from a normal distribution. The following null and research hypothesis were formulated.

H_o: There is no significant difference in mean between the X, Y and the Z coordinates of the two methods of survey.

H_1: That, there is a difference.

With the sample size equals 20, the paired t test was adopted.

To compute the test statistics, the standard deviation of the difference in coordinates of the two sample were calculated. A normal distribution and an unequal variance assumptions were made. The test was made under two significant levels of 0.05 and 0.1

The test statistics t is computed as:

$$t = \frac{\bar{d}}{s_{\bar{d}}}$$

where $\bar{d}$ represents the mean difference in X, Y, and Z, that is $\bar{d}_x$, $\bar{d}_y$ and $\bar{d}_z$,respectively.

$$\bar{d}_x = 0.17465$$

$$\bar{d}_y = 0.04455$$

$$\bar{d}_z = 0.01605$$

$n_1 = n_2 = 20$ with 19 degrees of freedom

For the X coordinates, the quantity d is calculated as (the coordinate obtained by total station - coordinate obtained by photogrammetry).

$$d = X_T - X_P$$

and the average difference is given as

$$\bar{d} = \frac{\sum_{x=1}^{20} d_x}{20}$$

The variance of the difference is computed as

$$s_d^{\,2} = \frac{\sum_{i=1}^{n}(d-\bar{d})^2}{n-1}$$

Using the above equations, the folowing data were obtained;

$$s_{d_x} = 0.125,\ s_{\bar{d}_x} = 0.03,\ s_{d_y} = 0.104,\ s_{\bar{d}_y} = 0.02,\ s_{d_z} = 0.089,\ s_{\bar{d}_z} = 0.02$$

where s_{d_x}, s_{d_y}, s_{d_z} and $s_{\bar{d}_x}$, $s_{\bar{d}_y}, s_{\bar{d}_z}$ are the standard deviations and the standard deviation of the means, respectively.

Performing a t test on these data, the following test statistics were computed

For the X coordinate,

$$t = \frac{0.1746}{0.03} = 5.82$$

For the Y coordinate,

$$t = \frac{0.04455}{0.02} = 2.2275$$

and for the Z coordinate,

$$t = \frac{0.01605}{0.02} = 0.8025$$

At 0.05 and 0.1 significant levels, The t values of 2.09 and 1.729 respectively rejects the null hypothesis on the X and Y coordinates but fails to reject that on Z. Thus the test reveals a difference in the X and Y coordinates of the two methods and no significant difference in the Z.

Although the test shows difference in the two coordinates, from table 9.3 one could see that for a point positioning of an accuracy of ±30cm, the average difference of 25cm between the methods indicates that any of the two methods could be used for data acquisition. But the point of interest will be the method that will be cost effective. The answer to this question is the point of discussion next.

9.3 Economic Analysis of the Survey Methods Used

In this analysis, the static GPS method is assumed to be the method used to establish the initial controls for all the other methods. Thus, it replaces the conventional survey method of traversing which might have been needed for Total station work.

It is also assumed that, the software cost and other petty logistical cost are not considered. It is also assumed that the 340 points used in making the contour map was surveyed by all three methods. The tables that follow explain away the cost and hours involved in the various methods. Areas where costing data were obtained are included in the analysis.

One independent model is assumed to have 114 points which is impractical.

Thus, it could be seen from the analysis that, the cost of survey varies with the method used. Using photogrammetric method is the most expensive while the total station method is the least expensive.

Table 9.4: Cost outline of GPS kinematic survey

Activity	Crew #	Hours spent	Total hours
GPS(static survey)			
T/D to SEPT	2	2.5	5
T/D to NEPT	2	2.5	5
SWPT to NWPT	2	1.5	3
Semi-kinematic survey	2	0.5[a]	1
Semi-kinematic. (wells)	2	8min/pt	90.1
Data processing. (static survey)	2	2	4
Data processing (semi-kin.)	1	3	6
Total hours			108.1

Total cost of using GPS equipment	5days x $140 = $700.00
Total labor cost	=108.1hrs. x $5 = $ 540.5
Total cost of survey	= $1240.50

[a] survey initialization, T/D represents "from Town and DOT".

Table 9.5: Cost outline of the Total station survey

Activity	Crew #	Hours spent	Total hours
GPS(Static)	2	5.5	11
Radial survey (Control points)	2	2.5	5
Radial survey (Wells)	2	4mins/pt.	45
Data proc.	1	3	3
Total hours			64

Cost of renting Total station equipment/day		= $75[a]
Cost of renting GPS equipment/day		= $140[b]
Labor cost per hour		= $5
Total cost of using GPS equipment	= 1day x $140	= $140.00
Total cost of using Total station equipt.	= 3days x $75	= $225.00
Total labor cost	= 64 hours x $5	= $320.00
Total cost of survey		= $685

[a] Iowa Engineers and Surveyors supply (4713 Lincolnway, Ames, Iowa)

[b] From Ashtech Inc.

Table 9.6: Aerotriangulation work

Activity	Crew #	Hours spent	Total hrs.
Model prep. and observation	1	11.3hrs/model	33.9
Data proc.	1	8	8
Total hours			41.9

Total cost of photography and photo processing		=$1200.00[a]
Total labor cost	= 41.9 x $5	=$209.50
Cost of setting field controls with the GPS and the total station		=$525.00
Total cost of photogrammetric aerotriangulation		=$1934.50

[a] Data obtained from Aerial services Inc., Davenport, IA.

Comparing the coordinates obtained from the kinematic survey and photogrammetry to that of the total station, it was evident that the differences are smaller with the GPS method. Thus, in relative sense, the GPS seems better than the Photogrammetric method.

A major problem with the GPS kinematic method was that as the distance between the rover station and the base station increases, the data acquired was error prone and was therefore useless. The problem is actually the inability to effectively fix the cycle slips which is commonly associated with the C/A code. Observation made with the P-code may not have similar problems. Another reason may be due to the receivers used. Also, as the PDOP exceeds 3, the positioning error increases.

CHAPTER 10

CONCLUSION AND RECOMMENDATIONS

10.1 **Conclusion**

The objective of developing a GIS for the portion of the Beer Creek in Roland using a combination of the total station, Photogrammetry, the global positioning system, and the GRASS GIS software was achieved. One could easily locate the wells by coordinates and also determine the general topography of the area using the contour values. However, the GRASS software is more of image processing than managing data and hence could not be used very extensively. The few queries that could be done were in the vector mode.

The use of the GPS and the total station in establishing controls for the study were invaluable. For without the the GPS, the whole work would have involved traversing to the nearest control point thereby increasing the total cost of the survey. The study revealed that obtaining the coordinates of points by radial survey using the total station is cost effective and simple.

Based on the overall work, the cost analysis and the statistical analysis, the following conclusions are drawn.

1. A base map covering the area of interest and having all the necessry features is an invaluable asset in any geographical information system.

2. All the three methods of data acquisition employed in the study area play a very significant role in GIS. It is very cost effective to establish the first few controls with static GPS survey and establish extra controls using Kinematic GPS or the total station. The choice however will depend on the size of the area in consideration. For a reasonably small area of say 1 square mile, a semi-kinematic GPS can be done very effectively. Otherwise, a total station is recommended for establishing extra controls as the semi-kinematic survey tends to be difficult as one have to carry the receiver-antenna configuration over long distances.

3. For any survey work whose positional accuracy is about $\pm$ 30cm, any of the three methods of photogrammetry, semi-kinematic survey and the total station may be used as the method of survey.

4. The cost of using Photogrammetric methods in acquiring data is more expensive than the GPS and the total station. The total station is the least expensive. However, in situations where the development of a GIS is involved, the Photogrammetry method may be cheaper as the photographs can be easily scanned and used in making the base map for the GIS.

5. Although, GPS is an all weather survey method, the system does not work well in the environment of other systems that communicate with electromagnetic waves like radios. Also, semi Kinematic GPS seems very useful especially when the points to survey are not far from each other. For points that are far from

each other, the semi-kinematic survey may not be very good especially when a single frequency C/A code receiver is used. Fixing the unavoidable cycle slips is a problem which results in unreliable integer ambiguities and hence the kinematic results not being very accurate.

10.2 **Recommendations**

From the experience and observations based on the study, the following recommendations are made;

1. The Global positioning system be used as the method of establishing the initial controls for any survey work by the static method and in cases where points which are far apart from each other (say one mile apart) are to be surveyed with the semi-kinematic mode, a dual frequency P-code receiver is recommended or the survey be done in pseudo-static or pseudo-kinematic mode.
2. Photogrammetry be used very actively in GIS especially in preparing base maps.
3. In determinig the coordinates of points which are not far from each other, the total station be adopted as the method of survey.
4. In digitizing a base map for GIS on AutoCAD, the use of the polyline option be done with extra caution as any single bit of line digitized is given an attribute which tends to make the cleaning process of the GIS development very laborious.

5. In a survey to compare the coordinates obtained with the total station and kinematic survey, the system be set up such that the total station reflector and the GPS antenna will be at the target station for simultaneous data collection. This will save time and also give the same weather effect on the data collection.
6. It is also recommended for further studies that the actual effects of different PDOP values and the presence of other electromagnetic recieving systems on point positioning be studied.
7. That a more data management GIS software like the ARC/INFO be used in developing such GIS

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APPENDIX A: COORDINATES OF POINTS IN THE THREE COORDINATE SYSTEMS USED

Spherical coordinates of points surveyed

Point No.	Latitude °	Latitude '	Latitude ''	Longitude °	Longitude '	Longitude ''
sept	42	11	18.246552	93	29	14.478005
nwpt	42	11	35.227820	93	29	32.051322
tspt	42	11	29.322858	93	29	25.559518
ept	42	11	26.773534	93	29	14.570386
cpt	42	11	26.228810	93	29	23.300307
wpt	42	11	26.129258	93	29	31.618274
swpt	42	11	18.070310	93	29	31.844382
spt	42	11	18.005416	93	29	23.534147
nept	42	11	35.673219	93	29	14.492588
npt	42	11	35.309601	93	29	23.526738
w20	42	11	33.024826	93	29	20.155482
w21	42	11	30.001190	93	29	18.993685
w22	42	11	30.005734	93	29	23.874475
w23	42	11	25.991242	93	29	28.794520
w24	42	11	24.535626	93	29	24.534969
w25	42	11	23.210769	93	29	25.185292
w26	42	11	23.434203	93	29	25.515612
w27	42	11	23.597819	93	29	25.598567
w28	42	11	23.676022	93	29	25.935273
w29	42	11	23.100725	93	29	27.017854
w30	42	11	23.415738	93	29	27.205822
w31	42	11	23.561878	93	29	27.205627
w32	42	11	23.750125	93	29	27.280517
w33	42	11	24.446378	93	29	28.206762
w34	42	11	24.476714	93	29	28.211813
w35	42	11	23.328609	93	29	28.321497
w36	42	11	22.909281	93	29	29.173021
w37	42	11	23.081468	93	29	29.463385
w38	42	11	23.235376	93	29	29.760247
w39	42	11	23.340725	93	29	29.966333
w40	42	11	22.545729	93	29	30.457609
w41	42	11	22.016687	93	29	31.779607
w42	42	11	22.052177	93	29	31.785966
w43	42	11	22.606923	93	29	31.880034
w44	42	11	26.239946	93	29	22.794876
w45	42	11	27.643916	93	29	23.239367
w46	42	11	28.445043	93	29	22.890832
w47	42	11	26.428884	93	29	19.884687
w48	42	11	28.739581	93	29	18.094660
w49	42	11	23.842142	93	29	14.444542
w50	42	11	23.255235	93	29	15.774703

UTM COORDINATES

POINT	X(meters)	Y(meters)
sept,	459757.6456	4670809.5481
nwpt,	459357.5885	4671335.6068
tspt,	459505.4329	4671152.6255
ept,	459757.0291	4671072.5535
cpt,	459556.7020	4671056.8996
wpt,	459365.9018	4671054.9271
swpt,	459359.2817	4670806.3991
spt,	459549.8822	4670803.3005
nept,	459760.3816	4671347.0318
npt,	459553.1168	4671337.0037
w20,	459630.0336	4671266.0919
w21,	459656.1459	4671172.6828
w22,	459544.2018	4671173.4650
w23,	459430.6435	4671050.2971
w24,	459528.0835	4671004.8402
w25,	459512.9326	4670964.0641
w26,	459505.3958	4670970.9989
w27,	459503.5221	4670976.0561
w28,	459495.8132	4670978.5125
w29,	459470.8805	4670960.9117
w30,	459466.6251	4670970.6523
w31,	459466.6555	4670975.1596
w32,	459464.9712	4670980.9755
w33,	459443.8501	4671002.5720
w34,	459443.7396	4671003.5083
w35,	459441.0200	4670968.1124
w36,	459421.4145	4670955.2917
w37,	459414.7852	4670960.6408
w38,	459408.0035	4670965.4269
w39,	459403.2954	4670968.7034
w40,	459391.8859	4670944.2487
w41,	459361.4697	4670928.1066
w42,	459361.3301	4670929.2020
w43,	459359.2712	4670946.3242
w44,	459568.2966	4671057.1765
w45,	459558.3503	4671100.5370
w46,	459566.4861	4671125.1998
w47,	459635.0787	4671062.6210
w48,	459676.5430	4671133.6535
w49,	459759.3990	4670982.1256
w50,	459728.7864	4670964.1982

Point No.	X (feet)	Y (feet)	Elevation (feet)
1	4924650	3531800	1052.01
2	4924650	3531900	1052.01
3	4924650	3532000	1052.01
4	4924650	3532100	1052.01
5	4924650	3532200	1059.74
6	4924650	3532300	1057.69
7	4924650	3532400	1048.29
8	4924650	3532500	1048.29
9	4924550	3531700	1050.99
10	4924550	3531800	1050.99
11	4924550	3531900	1050.99
12	4924550	3532000	1050.99
13	4924550	3532100	1050.99
14	4924550	3532200	1058.24
15	4924550	3532300	1058.24
16	4924550	3532400	1050.54
17	4924550	3532500	1047.64
18	4924450	3531700	1051.39
19	4924450	3531800	1051.39
20	4924450	3531900	1054.34
21	4924450	3532000	1054.34
22	4924450	3532100	1053.89
23	4924450	3532200	1055.39
24	4924450	3532300	1057.99
25	4924450	3532400	1054.74
26	4924450	3532500	1057.94
27	4924450	3532600	1051.49
28	4924350	3531700	1051.54
29	4924350	3531800	1051.54
30	4924350	3531900	1052.49
31	4924350	3532000	1052.49
32	4924350	3532100	1050.39
33	4924350	3532200	1055.79
34	4924350	3532300	1055.79
35	4924350	3532400	1048.54
36	4924350	3532500	1045.24
37	4924350	3532600	1039.24
38	4924250	3531700	1051.44
39	4924250	3531800	1054.99
40	4924250	3531900	1055.09
41	4924250	3532000	1048.79
42	4924250	3532100	1047.39
43	4924250	3532200	1043.49
44	4924250	3532300	1043.64
45	4924250	3532400	1043.69
46	4924250	3532500	1045.99
47	4924250	3532600	1039.39
48	4924150	3531700	1051.99
49	4924150	3531800	1054.89
50	4924150	3531900	1055.64
51	4924150	3532000	1045.09
52	4924150	3532100	1047.34
53	4924150	3532200	1042.99
54	4924150	3532300	1044.14
55	4924150	3532400	1044.14

APPENDIX B: COORDINATES OF GRID POINTS USED FOR PLOTTING THE CONTOURS

Point No.	X (feet)	Y (feet)	Elevation (feet)
1	4924650	3531800	1052.01
2	4924650	3531900	1052.01
3	4924650	3532000	1052.01
4	4924650	3532100	1052.01
5	4924650	3532200	1059.74
6	4924650	3532300	1057.69
7	4924650	3532400	1048.29
8	4924650	3532500	1048.29
9	4924550	3531700	1050.99
10	4924550	3531800	1050.99
11	4924550	3531900	1050.99
12	4924550	3532000	1050.99
13	4924550	3532100	1050.99
14	4924550	3532200	1058.24
15	4924550	3532300	1058.24
16	4924550	3532400	1050.54
17	4924550	3532500	1047.64
18	4924450	3531700	1051.39
19	4924450	3531800	1051.39
20	4924450	3531900	1054.34
21	4924450	3532000	1054.34
22	4924450	3532100	1053.89
23	4924450	3532200	1055.39
24	4924450	3532300	1057.99
25	4924450	3532400	1054.74
26	4924450	3532500	1057.94
27	4924450	3532600	1051.49
28	4924350	3531700	1051.54
29	4924350	3531800	1051.54
30	4924350	3531900	1052.49
31	4924350	3532000	1052.49
32	4924350	3532100	1050.39
33	4924350	3532200	1055.79
34	4924350	3532300	1055.79
35	4924350	3532400	1048.54
36	4924350	3532500	1045.24
37	4924350	3532600	1039.24
38	4924250	3531700	1051.44
39	4924250	3531800	1054.99
40	4924250	3531900	1055.09
41	4924250	3532000	1048.79
42	4924250	3532100	1047.39
43	4924250	3532200	1043.49
44	4924250	3532300	1043.64
45	4924250	3532400	1043.69
46	4924250	3532500	1045.99
47	4924250	3532600	1039.39
48	4924150	3531700	1051.99
49	4924150	3531800	1054.89
50	4924150	3531900	1055.64
51	4924150	3532000	1045.09
52	4924150	3532100	1047.34
53	4924150	3532200	1042.99
54	4924150	3532300	1044.14
55	4924150	3532400	1044.14

56	4924150	3532500	1043.24
57	4924150	3532600	1051.59
58	4924050	3531700	1053.19
59	4924050	3531800	1056.64
60	4924050	3531900	1049.94
61	4924050	3532000	1044.39
62	4924050	3532100	1044.39
63	4924050	3532200	1042.09
64	4924050	3532300	1042.09
65	4924050	3532400	1042.09
66	4924050	3532500	1042.09
67	4923900	3531700	1052.69
68	4923900	3531800	1058.64
69	4923900	3531900	1046.14
70	4923900	3532000	1044.74
71	4923900	3532100	1046.59
72	4923900	3532200	1038.54
73	4923800	3531700	1049.14
74	4923800	3532000	1043.24
75	4923800	3532100	1043.34
76	4923800	3532200	1039.39
77	4923800	3532400	1039.29
78	4923700	3531700	1049.74
79	4923700	3532200	1040.09
80	4923700	3532300	1038.79
81	4923600	3531700	1046.53
82	4923600	3531800	1043.74
83	4923600	3531900	1043.74
84	4923600	3532000	1043.74
85	4923600	3532100	1041.89
86	4923600	3532500	1041.84
87	4923600	3532600	1041.84
88	4923500	3531700	1042.84
89	4923500	3531800	1042.84
90	4923500	3531900	1042.84
91	4923500	3532000	1044.14
92	4923500	3532100	1041.94
93	4923500	3532200	1040.64
94	4923500	3532400	1038.24
95	4923500	3532500	1038.24
96	4923500	3532600	1038.24
97	4923400	3531700	1040.54
98	4923400	3531800	1040.54
99	4923400	3531900	1042.84
100	4923400	3532000	1042.84
101	4923400	3532100	1040.34
102	4923400	3532200	1043.64
103	4923400	3532300	1046.69
104	4923400	3532400	1046.69
105	4923400	3532500	1041.94
106	4923400	3532600	1041.94
107	4924670	3532570	1042.90
108	4924650	3532600	1044.60
109	4924650	3532700	1047.80
110	4924650	3532800	1048.70

111	4924650	3532900	1048.60
112	4924650	3533000	1042.00
113	4924650	3533100	1042.00
114	4924650	3533300	1044.50
115	4924550	3532700	1047.00
116	4924550	3532800	1047.00
117	4924550	3532900	1045.80
118	4924550	3533000	1036.80
119	4924550	3533100	1040.20
120	4924550	3533200	1040.30
121	4924550	3533300	1051.90
122	4924450	3532700	1044.90
123	4924450	3532800	1042.00
124	4924450	3532900	1034.30
125	4924450	3533000	1040.90
126	4924450	3533100	1040.80
127	4924450	3533200	1043.40
128	4924450	3533300	1053.50
129	4924350	3532600	1040.90
130	4924350	3532700	1043.00
131	4924350	3532900	1039.40
132	4924350	3533000	1039.90
133	4924350	3533100	1041.60
134	4924350	3533200	1051.80
135	4924350	3533300	1053.20
136	4924250	3532700	1038.00
137	4924250	3532800	1038.70
138	4924250	3532900	1040.00
139	4924250	3533000	1041.50
140	4924250	3533100	1041.90
141	4924250	3533300	1050.40
142	4924150	3532500	1041.20
143	4924150	3532600	1039.80
144	4924150	3532800	1040.40
145	4924150	3532900	1038.20
146	4924150	3533000	1042.00
147	4924150	3533100	1041.80
148	4924150	3533200	1043.70
149	4924050	3532500	1040.30
150	4924050	3532600	1039.10
151	4924050	3532700	1038.70
152	4924050	3532800	1035.00
153	4924050	3532900	1043.50
154	4924050	3533000	1046.50
155	4924050	3533100	1048.70
156	4924050	3533200	1047.90
157	4923950	3532500	1038.70
158	4923950	3532600	1039.90
159	4923950	3532700	1037.10
160	4923950	3532800	1043.70
161	4923950	3532900	1047.80
162	4923950	3533000	1047.80
163	4923950	3533100	1047.10
164	4923950	3533200	1047.40
165	4923850	3532500	1037.70

166	4923850	3532600	1038.30
167	4923850	3532700	1041.70
168	4923850	3532800	1044.10
169	4923850	3532900	1046.90
170	4923850	3533000	1046.40
171	4923850	3533100	1046.40
172	4923850	3533200	1047.10
173	4923750	3532500	1037.00
174	4923750	3532600	1038.90
175	4923750	3532700	1043.70
176	4923750	3532800	1042.10
177	4923750	3533000	1046.90
178	4923750	3533100	1045.70
179	4923650	3532500	1038.00
180	4923650	3532600	1040.30
181	4923650	3532700	1041.30
182	4923650	3533000	1047.50
183	4923650	3533100	1046.80
184	4923650	3533200	1049.30
185	4923650	3533300	1049.20
186	4923550	3532700	1043.70
187	4923550	3532800	1044.60
188	4923550	3532900	1047.90
189	4923550	3533000	1047.90
190	4923550	3533100	1047.90
191	4923550	3533200	1047.60
192	4923450	3532500	1039.40
193	4923450	3532600	1042.10
194	4923450	3532700	1045.50
195	4923450	3532800	1045.90
196	4923350	3532500	1039.70
197	4923350	3532600	1041.70
198	4923350	3532700	1045.20
199	4924650	3533350	1043.00
200	4924650	3533450	1051.40
201	4924650	3533550	1050.20
202	4924650	3533650	1050.20
203	4924650	3533750	1050.15
204	4924650	3533850	1053.03
205	4924650	3533950	1054.20
206	4924650	3534050	1054.20
207	4924650	3534150	1054.50
208	4924650	3534200	1054.70
209	4924550	3533350	1052.40
210	4924550	3533450	1051.00
211	4924550	3533550	1050.10
212	4924550	3533650	1050.10
213	4924550	3533750	1050.90
214	4924550	3533850	1051.70
215	4924550	3533950	1052.60
216	4924550	3534050	1054.90
217	4924550	3534150	1054.90
218	4924550	3534200	1055.00
219	4924450	3533300	1052.40
220	4924450	3533400	1051.90

221	4924450	3533500	1050.35
222	4924450	3533600	1049.70
223	4924450	3533700	1049.55
224	4924450	3533800	1051.90
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227	4924450	3534100	1054.70
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229	4924350	3533250	1051.40
230	4924350	3533350	1050.70
231	4924350	3533450	1049.40
232	4924350	3533550	1049.20
233	4924350	3533650	1050.25
234	4924350	3533750	1050.40
235	4924350	3533850	1051.45
236	4924350	3533950	1054.40
237	4924350	3534050	1058.00
238	4924350	3534150	1058.60
239	4924250	3533250	1048.50
240	4924250	3533350	1048.40
241	4924250	3533450	1048.55
242	4924250	3533550	1049.25
243	4924250	3533650	1050.30
244	4924250	3533750	1041.50
245	4924250	3533850	1034.80
246	4924250	3533950	1056.35
247	4924250	3534050	1056.90
248	4924250	3534150	1058.45
249	4924150	3533250	1046.15
250	4924150	3533350	1048.30
251	4924150	3533450	1048.70
252	4924150	3533550	1050.70
253	4924150	3533650	1052.30
254	4924150	3533750	1056.50
255	4924150	3533850	1055.90
256	4924150	3533950	1057.20
257	4924150	3534050	1057.80
258	4924150	3534150	1056.80
259	4924050	3533250	1046.20
260	4924050	3533350	1047.05
261	4924050	3533450	1047.90
262	4924050	3533550	1051.60
263	4924050	3533650	1055.80
264	4924050	3533750	1055.70
265	4924050	3533850	1055.60
266	4924050	3533950	1055.90
267	4924050	3534050	1056.40
268	4924050	3534150	1056.40
269	4923950	3533250	1046.00
270	4923950	3533350	1047.05
271	4923950	3533450	1049.50
272	4923950	3533550	1053.10
273	4923950	3533650	1056.80
274	4923950	3533750	1055.10
275	4923950	3533850	1062.20

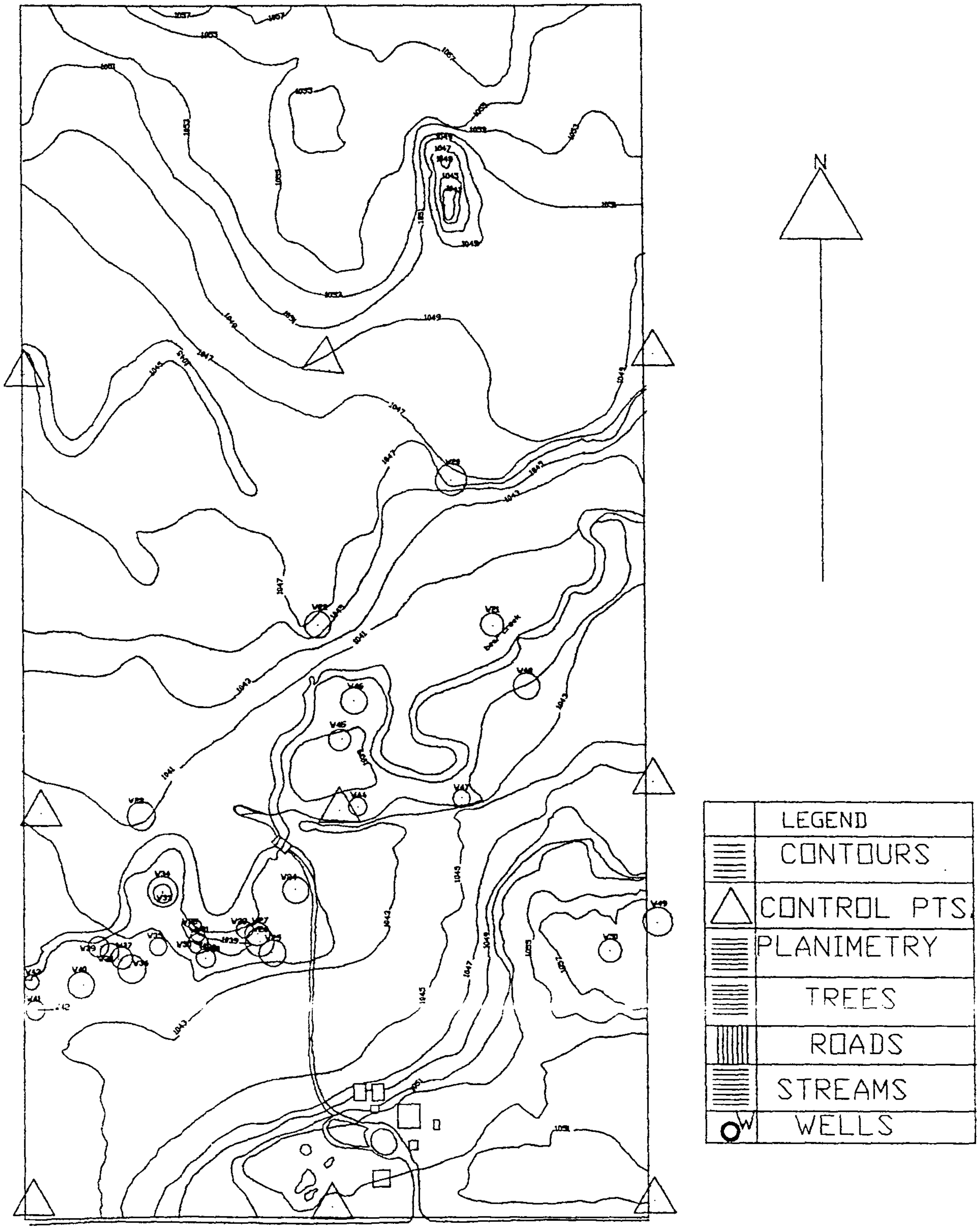
276	4923950	3533950	1054.10
277	4923950	3534050	1055.00
278	4923950	3534150	1058.70
280	4923850	3533300	1045.50
281	4923850	3533400	1048.60
282	4923850	3533500	1050.80
283	4923850	3533600	1051.90
284	4923850	3533700	1053.50
285	4923850	3533800	1053.75
286	4923850	3533900	1054.55
287	4923850	3534000	1054.80
288	4923850	3534100	1057.70
289	4923850	3534200	1060.10
290	4923750	3533200	1044.50
291	4923750	3533300	1044.70
292	4923750	3533400	1045.60
293	4923750	3533500	1048.75
294	4923750	3533600	1050.60
295	4923750	3533700	1049.80
296	4923750	3533800	1053.40
297	4923750	3533900	1052.20
298	4923750	3534000	1051.80
299	4923750	3534100	1053.65
300	4923750	3534200	1057.00
301	4923650	3533200	1044.50
302	4923650	3533300	1044.69
303	4923650	3533400	1044.90
304	4923650	3533500	1045.70
305	4923650	3533600	1048.00
306	4923650	3533700	1047.70
307	4923650	3533800	1049.35
308	4923650	3533900	1051.60
309	4923650	3534000	1051.50
310	4923650	3534100	1053.70
311	4923650	3534200	1059.60
312	4923550	3533200	1044.75
313	4923550	3533300	1043.30
314	4923550	3533400	1045.00
315	4923550	3533500	1045.30
316	4923550	3533600	1045.70
317	4923550	3533700	1047.20
318	4923550	3533800	1047.75
319	4923550	3533900	1049.35
320	4923550	3534000	1050.15
321	4923550	3534100	1053.70
322	4924680	3533470	1050.50
323	4923450	3533200	1044.80
324	4923450	3533300	1045.30
325	4923450	3533400	1045.20
326	4923450	3533500	1044.60
327	4923450	3533600	1046.80
328	4923450	3533700	1046.00
329	4923450	3533800	1047.10
330	4923450	3533900	1048.40
331	4923450	3534000	1049.70
332	4923450	3534100	1051.70
333	4923450	3534200	1053.30
334	4923350	3533300	1044.80
335	4923350	3533400	1044.80

336	4923350	3533500	1045.40
337	4923350	3533600	1046.50
338	4923350	3533700	1046.30
339	4923350	3533800	1046.30
340	4923350	3533900	1050.20
341	4923350	3534000	1053.30

APPENDIX C: HARD COPIES OF MAPS GENERATED

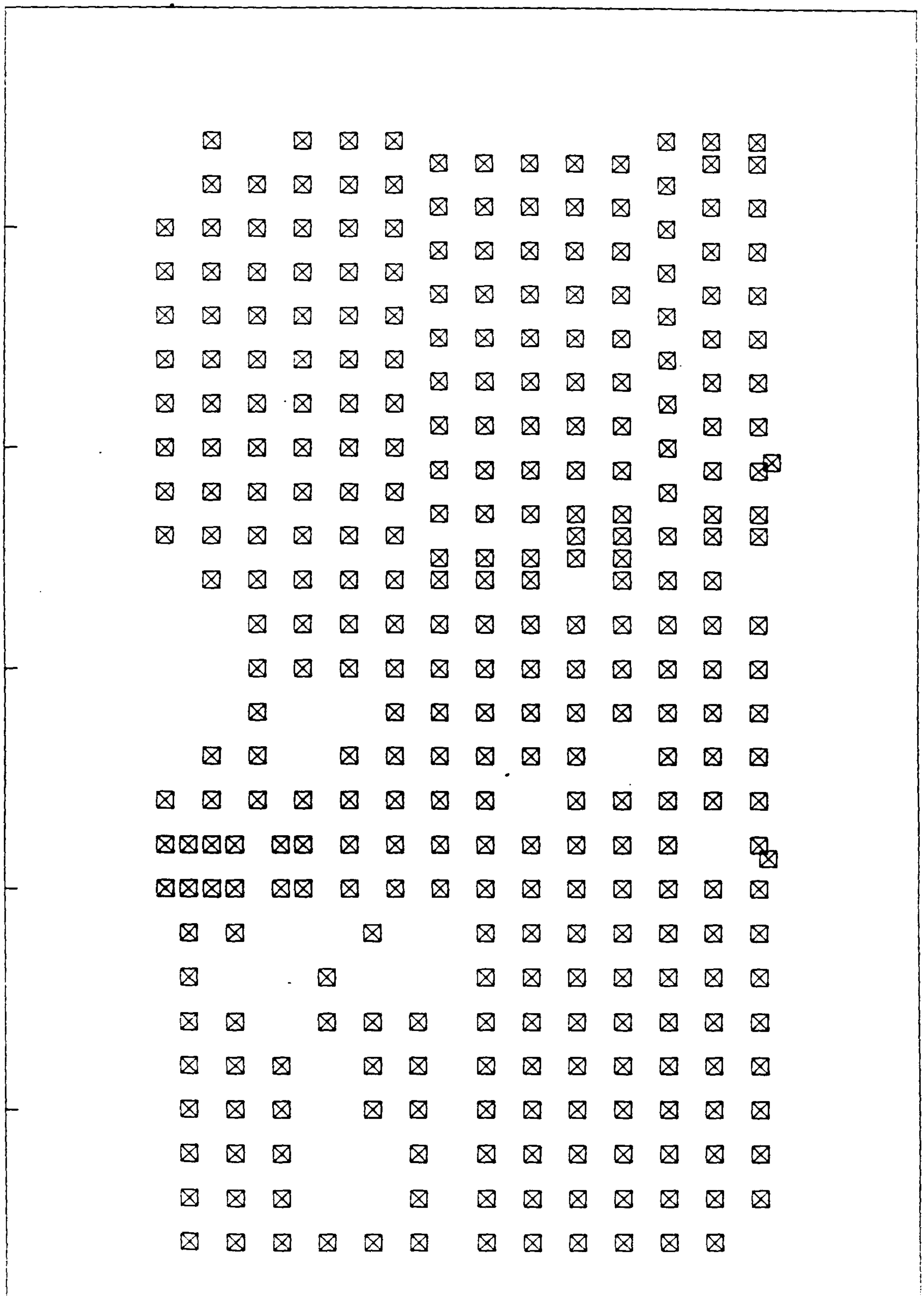
TOPOGRAPHIC MAP OF STUDY AREA

CONTOUR INTERVAL 2 FEET

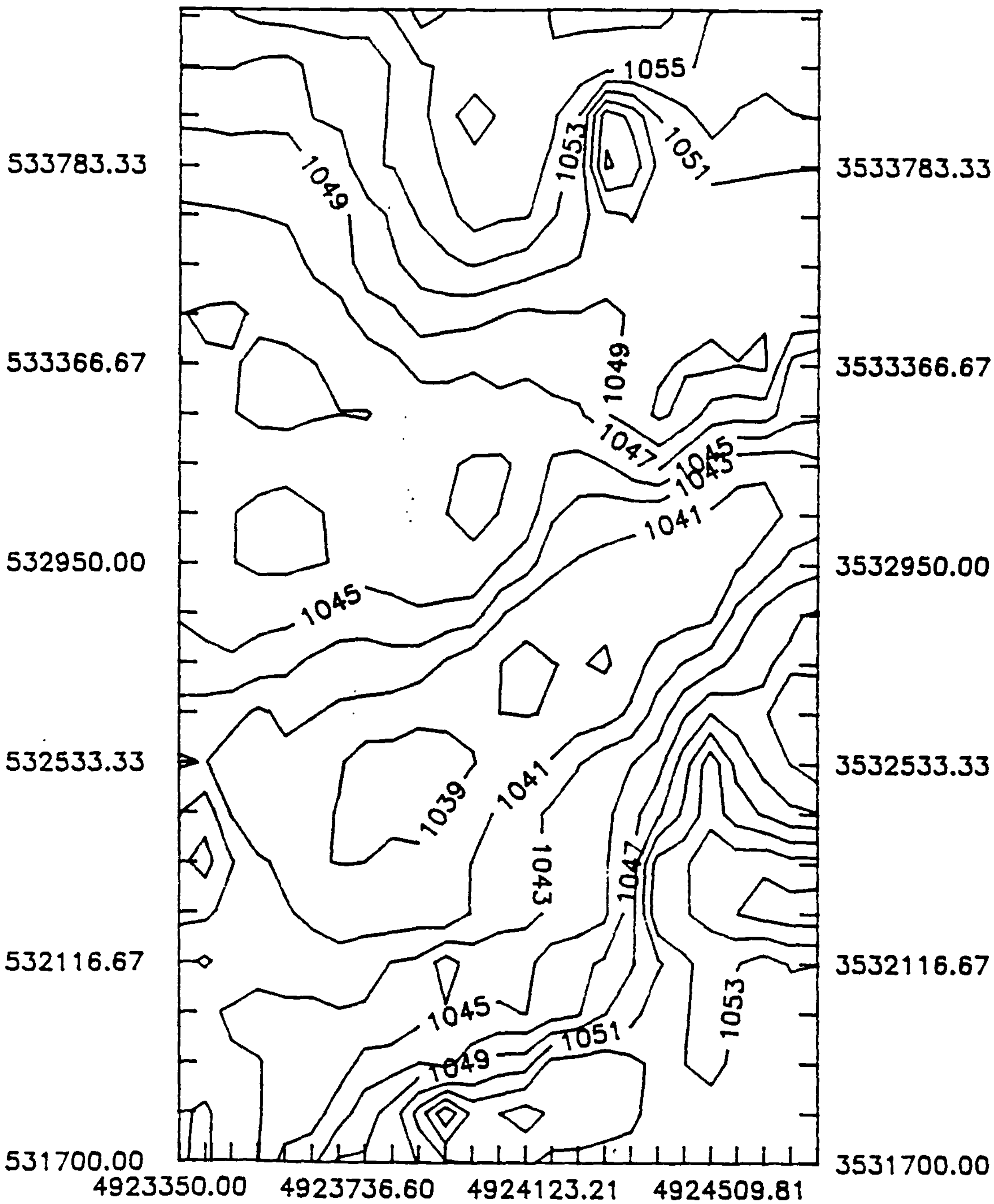


SCALE = 1:4300

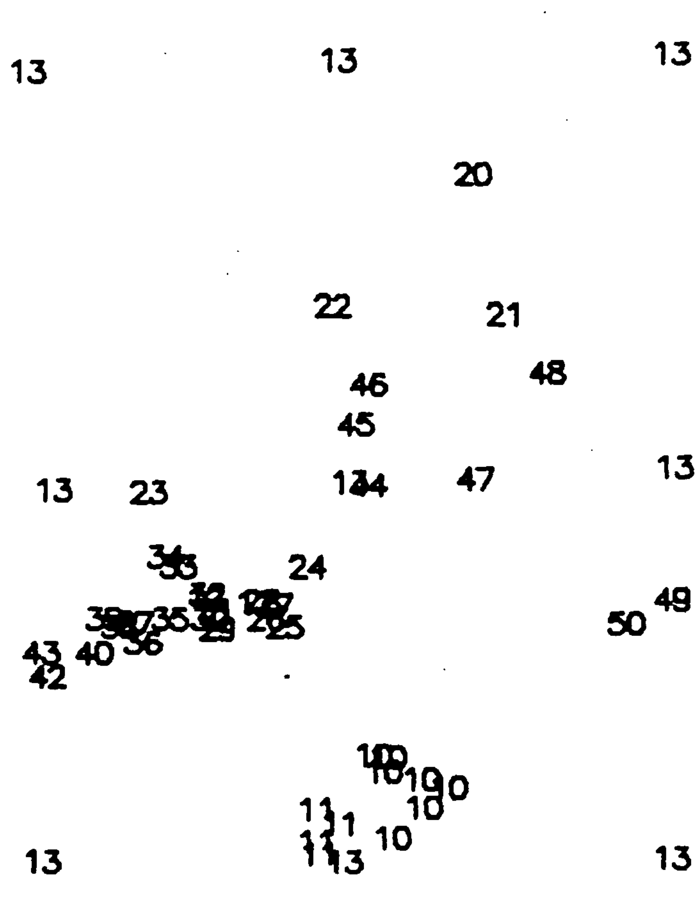
A PLOT OF POINTS USED BY SURFER IN GENERATING THE CONTOURS



CONTOUR MAP OF STUDY AREA
CONTOUR INTERVAL = 2FT



A PLOT OF A QUERY (IN GRASS) TO LOCATE THE RELATIVE LOCATIONS OF ALL THE WELLS



A PLOT OF A QUERY (IN GRASS) TO LOCATE THE POSITION OF ONE PARTICULAR WELL OF INTEREST