


2003

Establishment of a high accuracy geoid correction model and geodata edge match

Ruifeng Xi
Iowa State University

Follow this and additional works at: <https://lib.dr.iastate.edu/rtd>

 Part of the [Applied Mathematics Commons](#), [Civil Engineering Commons](#), [Earth Sciences Commons](#), [Environmental Monitoring Commons](#), and the [Remote Sensing Commons](#)

Recommended Citation

Xi, Ruifeng, "Establishment of a high accuracy geoid correction model and geodata edge match " (2003). *Retrospective Theses and Dissertations*. 755.
<https://lib.dr.iastate.edu/rtd/755>

This Dissertation is brought to you for free and open access by the Iowa State University Capstones, Theses and Dissertations at Iowa State University Digital Repository. It has been accepted for inclusion in Retrospective Theses and Dissertations by an authorized administrator of Iowa State University Digital Repository. For more information, please contact digirep@iastate.edu.

Establishment of a high accuracy geoid correction model and geodata edge match

by

Ruifeng Xi

A dissertation submitted to the graduate faculty
in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

Major: Civil Engineering (Geometronics)

Program of Study Committee:
Kandiah Jeyapalan, Major Professor
Dah-Yinn Lee
Say Kee Ong
Sunday Tim
Thomas Colvin

Iowa State University

Ames, Iowa

2003

Copyright © Ruifeng Xi, 2003. All rights reserved.

UMI Number: 3118269

INFORMATION TO USERS

The quality of this reproduction is dependent upon the quality of the copy submitted. Broken or indistinct print, colored or poor quality illustrations and photographs, print bleed-through, substandard margins, and improper alignment can adversely affect reproduction.

In the unlikely event that the author did not send a complete manuscript and there are missing pages, these will be noted. Also, if unauthorized copyright material had to be removed, a note will indicate the deletion.

UMI[®]

UMI Microform 3118269

Copyright 2004 by ProQuest Information and Learning Company.

All rights reserved. This microform edition is protected against unauthorized copying under Title 17, United States Code.

ProQuest Information and Learning Company
300 North Zeeb Road
P.O. Box 1346
Ann Arbor, MI 48106-1346

Graduate College
Iowa State University

This is to certify that the doctoral dissertation of

Ruifeng Xi

has met the dissertation requirements of Iowa State University

Signature was redacted for privacy.

Major Professor

Signature was redacted for privacy.

For the Major Program

TABLE OF CONTENTS

LIST OF FIGURES	ix
LIST OF TABLES	xiii
ACKNOWLEDGEMENTS	xv
ABSTRACT	xvii
CHAPTER 1 INTRODUCTION	1
1.1 GEOID Model	4
1.2 A Review Of Current Practices	6
1.2.1 CORS and HARN	6
1.2.2 GEOID Model	7
1.3 The Problems	9
1.4 Research Objectives	11
1.5 Thesis Organization	13

1.6 Summary and Conclusions	15
CHAPTER 2 GLOBAL POSITIONING SYSTEM	16
2.1 Introduction	16
2.1.1 Control Segment	17
2.1.2 Space Segment	18
2.1.3 User Segment	19
2.2 GPS Accuracy	20
2.2.1 Precise Positioning Service (PPS)	21
2.2.2 Standard Positioning Service (SPS)	23
2.3 GPS Satellite Signals	25
2.4 Pseudo-Range Navigation	26
2.5 Receiver Position, Velocity, and Time	30
2.6 SV Time, GPS Time, and UTC	31
2.7 Carrier Phase Tracking (Surveying)	32
2.8 Accuracy of Position and Time	34
2.9 Accuracy Limiting Factors	35
2.10 GPS Error Sources	36
2.10.1 Selective Availability (SA)	36
2.10.2 Other Bias Error sources	37

2.11 Geometric Dilution of Precision (GDOP) and Visibility	39
2.12 Differential GPS (DGPS) Techniques	41
2.12.1 Differential Code GPS (Navigation)	42
2.12.2 Differential Carrier GPS (Survey)	44
CHAPTER 3 HIGH ACCURACY REFERENCE NETWORK	45
3.1 Introduction	45
3.2 Project Definition	47
3.3 Continuously Operating Reference Stations (CORS) Network	50
3.3.1 Introduction	50
3.3.2 CORS Station Operators	57
3.3.3 Availability	54
3.3.4 File Naming Convention	60
3.4 CORS Positions and Velocities	61
3.4.1 CORS Data	61
3.4.2 Transforming Positions	66
3.4.3 Coordinate Revision	67
3.4.4 Transforming Velocities	68
3.5 Differential GPS Using CORS	69

3.5.1 No-Cost GPS Observations	
Available Nationally	69
3.5.2 Expected to Achieve 1 Centimeter Accuracy	71
3.5.3 Central Data Facility	73
3.5.4 Precise Orbital Information	74
3.5.5 CORS DGPS and HARN GPS Data	
Analysis Processing	75
CHAPTER 4. HARN GPS DATA PROCESSING	77
4.1 Introduction	77
4.2 Reformat Raw Data - RINGO	80
4.3 Merge a Database	80
4.4 Cycle Slip Fixing	88
4.4.1 Advantages of the Interactive Approach	88
4.4.2 The Steps of the Cycle Slip Fixing Process	90
4.5 GPS22	99
4.5.1 Final Solution of DGPS	99
4.5.2 Processing Mode	100
4.6 Fixing Integers	107
4.7 Adjustment of HARN Project	111

CHAPTER 5. HARN GEOID HEIGHT MODEL	126
5.1 Geoid Modeling Theory	126
5.2 The National Geoid Height Models	130
5.3 Iowa HARN GEOID Model	131
5.4 A Region GEOID Model	141
5.5 Model Test	152
CHAPTER 6. MODELING LOCAL GEOID WITH GPS AND LEVELING	159
6.1 Introduction	159
6.2 Principle of the Kalman Filter	164
6.3 Modeling of Local Geoid by Polynomial Surface Fitting	169
6.4 Basic Principles of the Least Squares Trend Analysis	172
6.5 Statistical Analysis of the Results	173
6.6 Local Geoid Model for Story County	176
6.7 Estimation of Unknown Polynomial Coefficients	186
6.8 Predication of Geoidal Heights	195
6.9 Local GEOID Model Analysis Conclusions	200
CHAPTER 7 RESULTS AND CONCLUSIONS	203

APPENDIX A GIS COVERAGE (GEODATABASE) EDGE MATCH	209
A.1 Introduction	209
A.2 Iowa Soil Project	210
A.3 Section Match Method	213
APPENDIX B SOIL SECTION MATCH AML	227
REFERENCES	259

LIST OF FIGURES

Fig. 1.1 Relationship of Ellipsoidal Height (h) and Orthometric Height (H)	4
Fig. 1.2 CORS Station and Data Rate	6
Fig. 1.3 GEOID96 Model for the United States	8
Fig. 2.1 GPS Segments	16
Fig. 2.2 Locations of Control Segment Installations	17
Fig. 2.3 Functions of the Control Segment	18
Fig. 2.4 GPS Satellites Constellations	19
Fig. 2.5 PPS and SPS GPS Receiver Capabilities / Limitations	24
Fig. 2.6 Comparison of Position with and without Selective Availability	24
Fig. 2.7 ECEF X, Y, and Z	27
Fig. 2.8 Theoretically Three Satellites Could Position a Receiver	28
Fig. 2.9 Four Satellites Solution	29

Fig. 3.1 Iowa HARN Project	47
Fig. 3.2 Iowa HARN Points Control Areas	49
Fig. 3.3 GPS CORS Stations	50
Fig. 3.4 HARN GPS Data Analysis Procedure	75
Fig. 4.1 OMNI Data Processing Steps	78
Fig. 4.2 Data Merge Setup	81
Fig. 4.3 Plot the Satellite Elevation Plot	86
Fig. 4.4 The Steps of the Cycle Slip Fixing Process	91
Fig. 4.5 Correcting the Cycle Slips	96
Fig. 4.6 Fixing Cycle Slips	97
Fig. 4.7 Generating the Final IONO-free Solution	101
Fig. 4.8 Iowa HARN Project	114
Fig. 4.9 Major of Axis of HARN Project	116
Fig. 4.10 Vertical Accuracy of HARN Project	116
Fig. 4.11 Relative Accuracy of All Stations	117
Fig. 4.12 Distance Relative Precision of All Stations	118
Fig. 5.1 Iowa GEOID96 Model	132
Fig. 5.2 Iowa Leveling Points	133
Fig. 5.3 Normal Probability Plot	139
Fig. 5.4 The Differences between GPS Heights and Leveling Heights	140

Fig. 5.5 The Difference Frequency Chart	149
Fig. 5.6 Height Differences between GPS and Leveling	143
Fig. 5.7 Iowa Leveling Points Control Area	144
Fig. 5.8 The Leveling Points order and Control Area	145
Fig. 5.9 Effect Area of Different Elevation between GPS and Leveling	146
Fig. 5.10 Height Difference Between GPS and Leveling	147
Fig. 5.11 GPS Heights of Iowa and Around CORS Points	155
Fig. 5.12 Difference between Day85 and Day258	156
Fig. 5.13 Difference between Day282 and Day85	157
Fig. 5.14 Difference between Day78 and Day282	157
Fig. 6.1 A typical GPS Positioning Plan	164
Fig. 6.2 The Relationship between GPS Height and Orthometric Height	170
Fig. 6.3 Geodetic Control Network	177
Fig. 6.4 GEOID99 model for Story County Area	179
Fig. 6.5 The Difference between GEOID99 and GEOIOD96	181
Fig. 6.6 The Difference between Day85 and Day285	182
Fig. 6.7 The Difference between Day282 and Day85	183
Fig. 6.8 The Difference between Day78 and Day282	183
Fig. 6.9 The Difference between Day85 and Day285	184

Fig. 6.10 Polynomial Fitting Residuals for Each Observed Day	197
Fig. A.1 Rectified Aerial Photography Base Maps	212
Fig. A.2 Iowa Soil GIS Coverage Map	214
Fig. A.3 Comparison between PLS Match Method	216
Fig. A.4 Original Coverage	217
Fig. A.5 Match River Section with New Method	217
Fig. A.6 Match to PLS by Old Method	218
Fig. A.7 Match by New Method	218
Fig. A.8 Compare between PLS Match Method	219
Fig. A.9 Section Match to PLS Principal	220

LIST OF TABLES

Table 3.1 National CORS Station Operators	52
Table 3.2 CORS Station Data Availability	54
Table 4.1 Position Accuracy of HARN Project	118
Table 5.1 The Difference between GPS Height and Orthometric Height	133
Table 5.2 Statistic Analysis Results	137
Table 5.3 GPS Height and Orthometric Height	147
Table 5.4 Iowa HARN Geoid Correction Model	151
Table 5.5 Region Geoid Model Test Results	153
Table 5.6 GPS Results for CORS Stations around Iowa	154
Table 5.7 GPS Height Difference for CORS Stations around Iowa	156
Table 6.1 Story County Geoid Network Results	183
Table 6.2 The Orthometric Heights from GEOID99	185
Table 6.3 The Difference between GPS Converted Orthometric Heights and Geodetic Leveling Heights	187

Table 6.4 The estimated polynomial coefficients for Day258, 2000	194
Table 6.5 The Estimated Polynomial Coefficients for Day85, 2001	194
Table 6.6 The Estimated Polynomial Coefficients for Day282, 2001	195
Table 6.7 The Estimated Polynomial Coefficients for Day78, 2002	195
Table 6.8. Kalman Filter Model for Day285,Day85,Day282,Day78	198
Table 6.9 The Residuals of Kalman Filter Predicted and Filtered value	203

ACKNOWLEDGEMENTS

I would like to express my sincere appreciation for the support and assistance given by my major Professor, Dr. Kandiah Jeyapalan. His guidance and contributions have been vital both in the research and writing of this thesis, and in the rest of my graduate studies at Iowa State University. He has a natural ability to express complex ideas in easily comprehensible terms. I would also like to thank my Committee member Dr. Say Kee Ong, Dr. Dah-Yinn Lee, Dr David Kao, Dr. U. Sunday Tim, Dr. Thomas Colvin, and Dr. John Dobson for their assistance and in reviewing the manuscript.

My thanks also go to Kevin Kane, Director of ISU GIS Support and Research Facility, Dr. Edward Kannel, Patrick Brown, Marc Rogers, and Sharon Ferris for their support and assistance in this project and my studies at ISU. My thanks also go to Ramesh A Comondoor for his help. This research was made possible by support from the Iowa Department of Transportation, and ISU GIS Support

and Research Facility. The data used in this thesis were provided by Iowa DOT and ISU GIS Support and Research Facility.

Last, and certainly by no means least, are the thanks due to the other graduate students in the department who helped me in my studies and above all were there to keep my spirits up when things weren't looking so great. This task would have been immeasurably harder without their encouragement and support.

On a more personal note, this list of those who provided support and encouragement would not be complete without mention of family and friends; my wife Fangyuan Yang who has been there throughout my graduate studies as a friend and confidante, my daughter for her understanding, my parents who have provided support and encouragement, and everyone else who has been there for me. Thanks.

ABSTRACT

This research has developed a theoretical and practical methodology for efficiently and accurately determining sub-decimeter level regional geoids and centimeter level local geoids to meet regional surveying and local engineering requirements. This research also provides a highly accurate static DGPS network data pre-processing, post-processing and adjustment method and a procedure for a large GPS network like the state level HRAN project. The research also developed an efficient and accurate methodology to join soil coverages in GIS ARE/INFO.

A total of 181 GPS stations has been pre-processed and post-processed to obtain an absolute accuracy better than 1.5cm at 95% of the stations, and at all stations having a 0.5 ppm average relative accuracy. A total of 167 GPS stations in Iowa and around Iowa have been included in the adjustment.

After evaluating GEOID96 and GEOID99, a more accurate and suitable geoid model has been established in Iowa. This new Iowa regional geoid model improved the accuracy from a sub-decimeter 10~20 centimeter to 5~10 centimeter. The local kinematic geoid model, developed using Kalman filtering, gives results better than third order leveling accuracy requirement with 1.5 cm standard deviation.

CHAPTER 1 INTRODUCTION

The Global Positioning System (GPS) has been widely used in all positioning fields. Its importance can hardly be overstated. It has revolutionized surveying, mapping, navigation, and Geographic Information Systems (GIS). In particular, GPS now provides surveyors and engineers the capability of making geodetic measurements with an accuracy that, in the past, could only be obtained by geodetic surveys. Further, this new accuracy is achieved with greater efficiency and economy than was ever possible before GPS.

The GPS is three-dimensional, providing horizontal and vertical dimensions of a point in space. A GPS position is computed relative to a simple ellipsoidal model of the earth. The position accuracy depends on the receiver's ability to accurately calculate the time it takes for each satellite signal to reach earth. The primary error sources which can affect the position computations are ionosphere and troposphere delays on the radio signal, signal multi-path, receiver clock biases,

orbital errors, and the intentional degradation of the satellite signal by the Department of Defense (DOD).

The intentional degradation of the signal is known as "Selective Availability (SA)" and is intended to prevent adversaries from exploiting highly accurate GPS signals and using them against the United States or its allies. However, on May 1, 2000, U.S. President Bill Clinton ordered Selective Availability (SA) turned off at midnight (Coordinated Universal Time). Now, civilian GPS users around the world no longer experience the up to 100 meter (approximate 300 feet) random errors that SA added to keep GPS as a more powerful tool for military use. Today, GPS civil units are getting more accuracy in position determination than before. The remaining errors can be reduced or eliminated through a technique known as "Differential GPS (DGPS)". The idea of DGPS is that most errors at two receivers in close proximity to each other will be the same. DGPS will cancel most of errors in the DGPS vector computation. If we know the position of one receiver, we can get a very accurate position of another receiver by the DGPS vectors.

The carrier-phase tracking differential is obtained by comparing the carrier phases received by reference and remote receivers simultaneously. To correctly estimate the number of carrier

wavelengths at the reference and remote receivers, they must be close enough to insure that the ionospheric delay difference is less than the carrier wavelength. This usually means that carrier-phase GPS measurements must be taken with remote and reference stations within about 30 to 100 kilometers of each other. So, it is very important to have a high accuracy reference network to carry out DGPS around the area being surveyed. To use GPS as a geodetic control method, one needs to carefully control the error sources and choose the right method of collecting and processing data. A very important issue when using DGPS in engineering applications is how to transfer GPS heights to orthometric heights.

A High Accuracy Reference Network (HARN) for the State of Iowa giving horizontal dimension obtained from GPS has been established by National Geodetic Survey (NGS) in cooperation with Iowa Department of Transportation (IADOT). This research project develops a methodology to demonstrate how DGPS can be used to accurately determine horizontal and vertical position for engineering applications.

The following subsection discusses GEOID and how a GEOID model converts GPS heights to orthometric heights. Current practices of GEOID in GPS heights transformation are described and then problems related to Iowa HARN project are identified and resolved.

1.1 GEOID Model

The GPS is three-dimensional, providing heights and horizontal positions. The GPS heights, called ellipsoidal heights, are computed relative to a simple ellipsoidal model of the earth (Fig.1.1).

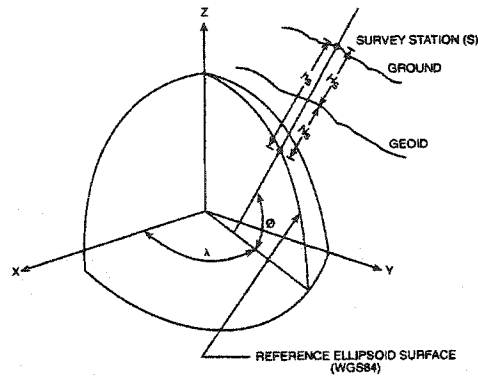


Fig. 1.1 Relationship of Ellipsoidal Height (h) and Orthometric Height(H)

However, heights found on topographic maps, engineering and GIS applications are related to the gravity field of the earth. We call them orthometric heights, which are the heights from mean sea level. Mean sea level is a physical surface. It is meaningful, but it is difficult to compute all coordinates on it. The geoid height N is the difference

between the ellipsoidal height and the orthometric height. These height systems are related by the equation:

$$h = H + N$$

Geoid heights allow one to accurately relate the heights from GPS surveys to heights above mean sea level. Without a Geoid to link the two heights surveyors cannot get the orthometric height from GPS, which is needed for engineering applications. Geoid height models help to supply the third dimension to GPS. In order to transfer high accurate GPS heights to orthometric heights with satisfying accuracy, we need a high accurate geoid model. **Without an accurate geoid model surveyors can only get a highly accurate 2D position from 3D GPS.**

The transformation accuracy depends on the accuracy of the geoid model. In the conterminous United States, geoid heights range from a low of -52.8 meters in the Atlantic to a high of -7.7 meters in the Rocky Mountains.

1.2 A Review Of Current Practices

1.2.1 CORS and HARN

The Continuously Operating Reference Station (CORS) GPS network coordinated by the National Geodetic Survey (NGS), National Oceanic and Atmospheric Administration (NOAA), is a group of GPS reference stations which provide code range and carrier phase data to users in support of post-processing DGPS applications nation wide. Government, academic, commercial and private users are given this information to determine after-the-fact positioning of fixed points and

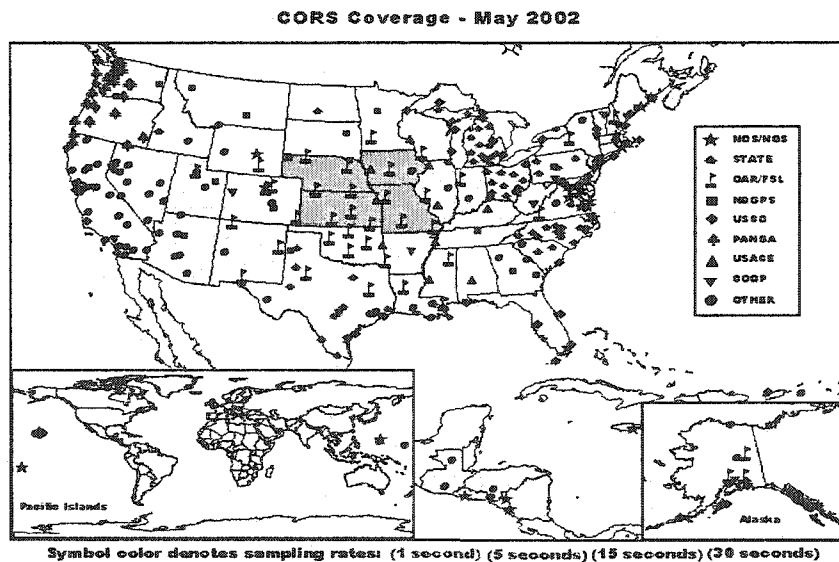


Fig. 1.2 CORS Station and Data Rate [149]

moving platforms. Currently, the CORS network consists of near by 300 stations located nationwide; a number that is continually increasing.

The GPS data are being recorded at a 30,15,5 or 1 second sampling rate in the Receiver INdependent EXchange (RINEX 2) format, version 2. The data sets are available free of charge and can be retrieved over the INTERNET at <http://www.ngs.noaa.gov/CORS>. The data are also available on CD ROM and can be purchased. Because of the cost of operation, CORS cannot provide DGPS services for all engineering applications. To get an accurate DGPS, we need a state wide high accuracy reference network.

1.2.2 National GEOID Model

The GEOID96 (Fig. 1.3) and GEOID99 models were published by the National Geodetic Survey in 1996 and 1999 for national geoid correction. Both GEOID models were developed to support direct conversion between ellipsoid heights expressed in the NAD 83 (86) reference frame and orthometric heights expressed in the NAVD 88 vertical datum. Due to the non-geocentricity of the NAD 83 (86), coupled with a probable constant bias in the definition of

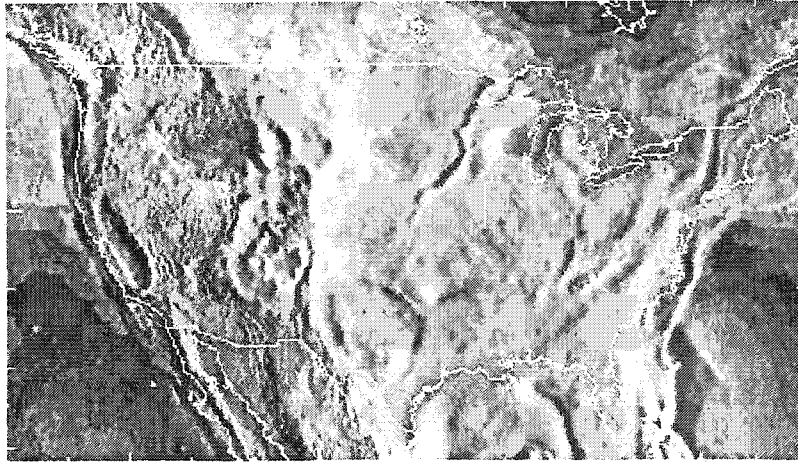


Fig.1.3 GEOID96 Model for the United States[156]

NAVD 88, we find discrepancies of a meter or more between a geocentric geoid model, such as GEOID96 or GEOID99, and GPS heights on leveled benchmarks. The difference between GEOID96 and GEOID99 is that GEOID99 is an improved version of GEOID96. There are better data and better computational methods in GEOID99. By means of known reference frame relationships, and 2951 GPS benchmark points in GEOID96 and 6169 GPS benchmark points in GEOID99, a smoother conversion surface was developed for the GEOID99 model than for the GEOID96 model.

1.3 The Problems

This research gives the accuracy of the GEOID96 model for the GPS benchmarks in Iowa. The differences using GEOID96 and GEOID99 at 81 GPS benchmarks are compared.

First, the GPS benchmark data set used to compute these empirical statistics is not the data set used to develop the conversion surface of GEOID96. Second, the GEOID96 empirical statistics are averages over the entire state of Iowa. Therefore, one should examine the geoid correction result accuracy for Iowa before using it. GEOID96 may not suit the geodetic and engineering applications for all of Iowa. If so, we should develop a better and more accurate geoid model for the state of Iowa and supplement it with a local model for a small local area.

The estimated accuracy for Iowa HARN project using the CORS-fixed GPS adjustment ranged from a 0.97 to 1.96 cm in the horizontal direction, and 9.13 cm in the adjusted ellipsoid heights obtained from adjusted residual analysis. The average horizontal accuracy of the ellipsoid major axis is 0.0196, and minor axis is 0.0097. This means that GEOID96 and its conversion surface are not perfect. More accurate GPS ellipsoid heights on leveled benchmark data at a much

finer (~20 km or less) spacing would be required to resolve any hypothetical fine-scale geoid error. By definition, for engineering application, the national geoid model does not have the required accuracy. A local geoid model is necessary. The accuracy of a GEOID model depends on the accuracy required by the application.

Using HARN, we can get an accurate geo-reference for GIS application. One of these GIS applications in Iowa is the soil project. Iowa has 99 counties containing 1671 townships and more than 200,000 sections. Because of the map scale, scanner size, and other factors, all original digital coverage is on a section by section basis. To get township and county size soils coverage and a standard boundary section coverage by joining these section base coverages together, we must match section base coverage to Public Land Survey System (PLSS) coverage.

Arc/Info, a GIS software, offers a basic adjustment function, "adjust", to do this. For a single coverage, you can adjust it to a snapcoverage manually in Arcedit. However, for more than 200,000 coverages, it will be impossible to adjust them, one by one, manually. If one coverage takes one hour to adjust, 200,000 coverages will take about 70 GIS technicians one year to complete these adjustments. This is only the first step for creating GIS soils database on a township or

county size base. After lines, points and polygons are matched, we have to combine features from two or more coverages into a single output coverage. Even so, this does not calculate new topological relationships between the resulting features.

Before appending all section coverages to a township or county size coverage, we must match each section coverage to PLSS to get a common boundary. There is a sample method with Arc/Info Macro, developed by Iowa DNR to do this. However this method with Macro, AML (Arc/Info Macro Language) can only do a standard section match. It uses tic match method and distance search algorithm. It can only match less than 60% of a standard rectangle section, and less than 10% of a township. This research develops a method for overcoming these deficiencies and which can also be used to match local geoid correction developed county by county.

1.4 Research Objectives

The objective of this research is to develop a methodology for efficiently and accurately establishing and analyzing a DGPS reference network in both horizontal and vertical directions for state level and local areas. This 3D reference network and model should supply

accuracy and efficiency in engineering applications. The methodology includes several components, which are described below.

1. Theories of designing and establishing a HARN project, a DGPS data processing system and a system for analyzing GPS errors to fix cycle slips and their effects will be presented. Investigating and testing data, determining observation rejection and re-observation are also included here.

2. Theories of GPS network adjustment and adjustment of HARN, and the analysis of adjusted results are described.

3. An accuracy evaluation of the GEOID96 model for the state of Iowa establishing orthometric heights for a control network from GPS surveys are developed. This requires a highly accurate geoid model. This research would not only assess the national geoid model for Iowa, but would also provide diagnostic information on the GPS network and level network accuracies, and indicate future survey requirements for Iowa. After evaluating GEOID96 and GEOID99, a more accurate and suitable geoid model has been established in Iowa using both GPS and geodetic leveling information through out whole of Iowa. In this way, the Iowa HARN would serve both horizontal and vertical control needs, and maximum economic benefit could be obtained from the existing Iowa HARN project.

4. During the past two years, four sets of data have been collected at 11 points in Story County. This research developed a methodology to establish a high accuracy local geoid model for a local area to meet engineering applications.

5. In order to overcome problems in GIS edge match and associated “database join”, this research develops a new algorithm, which is called Section Match to PLSS (SMP). It can not only match a normal section to PLSS, it also can completely match river boundaries to PLSS. Even for large offsets from PLSS, which sometimes measures up to 200 meters, the new method gets very good results.

1.5 Thesis Organization

The fields of research relevant to this study include DGPS, Adjustment, GEOID, and GIS. In chapter 2, main GPS error sources, GPS surveying and data processing methods have been described.

Chapter 3 will discuss the Iowa HARN project, its design and its practical problems. In chapter 4, the processing of HARN data for this project will be described. The research also discusses HARN adjustment and gives the results and analysis of Iowa HRAN project. In order to achieve a more homogenous GPS HARN results, the A-order,

B-order, and First-order GPS surveys constituting the Iowa HARN are combined in a special adjustment. In this computation CORS and A-order stations are held fixed.

In chapter 5, the research begins with a review of the characteristics and computational procedures employed for the GEOID96 geoid height models. This review is particularly important, because this geoid model is used as part of Iowa height data set analysis. Recent results on formal and empirical geoid accuracy estimates are presented, followed by remarks on the origin of the GEOID96 accuracy estimates for the United States. Then, the geoid model accuracy is analyzed for the state of Iowa. This is performed with results from the Iowa NRAN data adjustment. The results are described and the various recommendations and possibilities for network improvement are presented. In chapter 6, a more accurate and more realistic local geoid model is established for Story County to meet engineering applications. Results and conclusions are included in chapter 7. The methodologies and results for the GIS coverage edge match and joining are discussed in Appendix A, and main AML programs are included in Appendix B.

1.6 Summary and Conclusions

This research develops a theoretical and practical methodology for efficiently and accurately determining a sub-decimeter level region geoid and a centimeter level local geoid to meet regional surveying and local engineering requirements. This research also provides a very accurate method of preprocessing, post-processing DGPS network data and adjustment procedure for a large GPS network like the state level HRAN project. The research also developed an efficient and accurate methodology to join soil coverages in ARE/INFO (See Appendix). The research can be used to join local geoid contours of adjoining counties. A detailed account of this method call "Section Match" is given in the Appendix.

During the past three years, four sets of data have been collected at 11 points in Story County. All these data has been processed by DGPS with precise ephemeris. This research developed a Kalman filtering methodology to establish a high accuracy local geoid model for a local area to support engineering applications. The methodology has been tested by these real data. The prediction gives a better than third order leveling accuracy and 1.5 cm standard deviation.

CHAPTER 2 GLOBAL POSITIONING SYSTEM

2.1 Introduction

The Global Positioning System (GPS) is funded and controlled by the U. S. Department of Defense (DOD). While there are many thousands of civil users of GPS worldwide, the system was designed for and is operated by the U. S. military

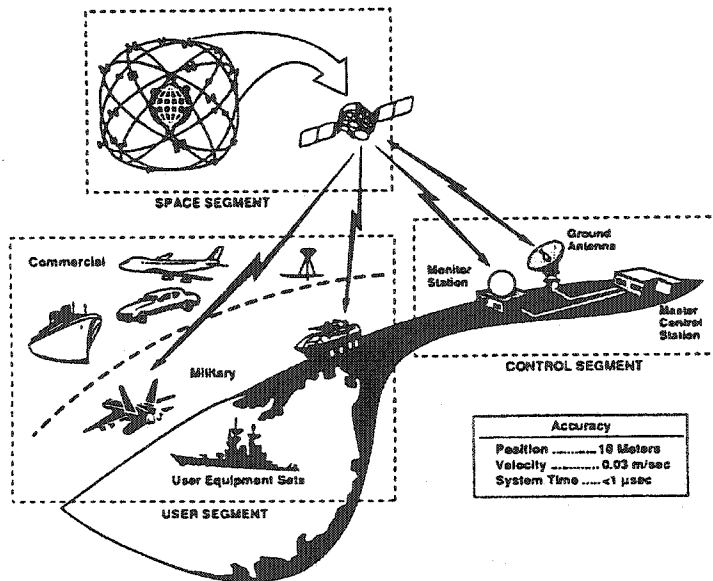


Fig.2.1 GPS segments [7]

Figure 2.1 shows that GPS consists of three segments: the space segment, the control segment and the user segment.

2.1.1 Control Segment

The Control Segment consists of a system of tracking stations located around the world. The Master Control Station (MCS) is located at Falcon Air Force Base (AFB) in Springs, Colorado. Five Monitor Stations (MS) are situated at Hawaii, Kwajalein, Diego Garcia, Ascension, and collocated with MCS at Falcan AFB (Fig.2.2). These stations measure signals from the space vehicles (SVs), which are incorporated into orbital models for each satellite. The models compute

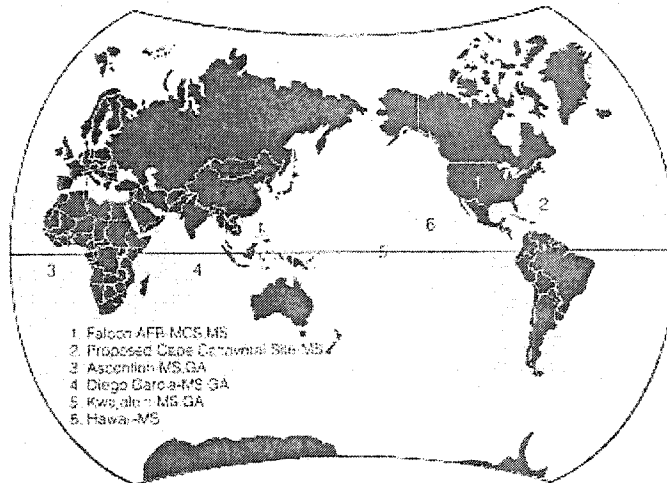


Fig. 2.2 Locations of Control Segment Installations[7]

precise orbital data (ephemeris) and SV clock corrections for each satellite. The Master Control Station uploads ephemeris and clock

data to the SVs. The SVs then send subsets of the orbital ephemeris data to GPS receivers over radio signals (Fig. 2.3).

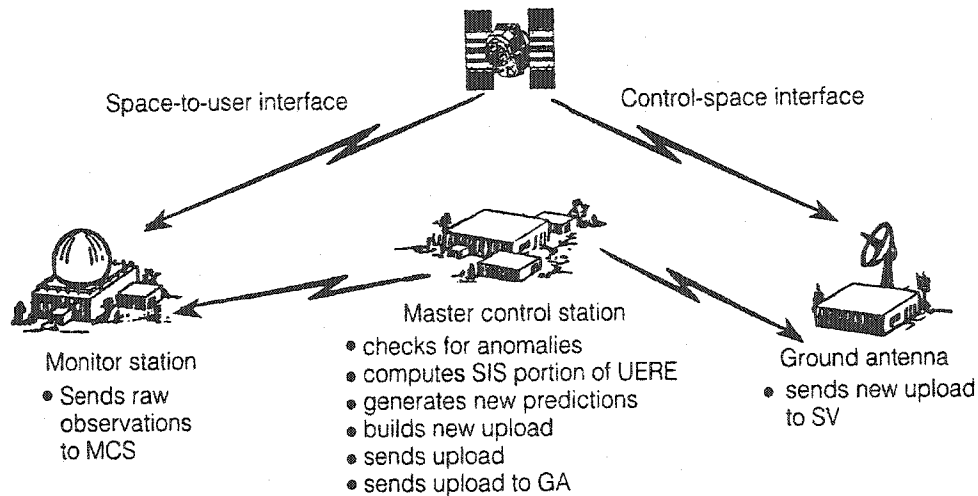


Fig. 2.3 Functions of the Control Segment [7]

2.1.2 Space Segment

The Space Segment of the GPS system consists of the GPS satellites. These space vehicles (SVs) send radio signals from space.

The GPS Operational Constellation consists of 24 satellites. The entire contingent of GPS satellites is in six orbit planes, with three or four operational satellites in each orbit plane. The orbital planes of the satellites have an inclination of 55 degrees (relative to the equator).

These orbits repeat the same ground track (as the earth turns beneath

them) once each day. The orbit altitude, which is 20,200km (10,900 miles), is such that the satellites repeat the same track and configuration over any point approximately every 24 hours (4 minutes earlier each day). The satellites are positioned in such a manner that a minimum of five satellites is normally observable by a user anywhere on the earth at any given time (Fig. 2.4).

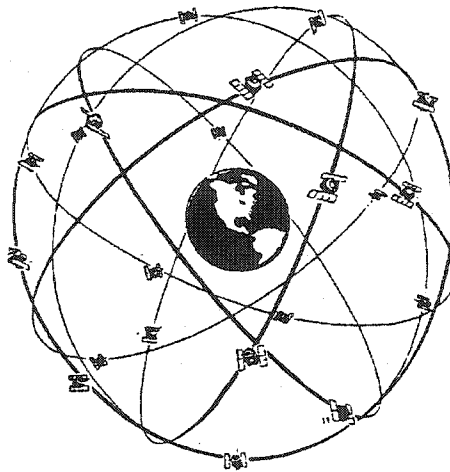


Fig. 2.4 GPS Satellites Constellations [7]

2.1.3 User Segment

The GPS User Segment consists of the GPS receivers and the user community. GPS receivers convert SV signals into position, velocity, and time estimates. Four satellites are required to compute the four dimensions of X, Y, Z (position) and Time. GPS receivers are

used for navigation, positioning, time dissemination, and other research.

GPS has been used in many application and research areas. Navigation in three dimensions is the primary function of GPS. Navigation receivers are made for aircraft, ships, ground vehicles, and for hand carrying receivers by individuals. Precise positioning is possible using Differential GPS (DGPS) method, which sets GPS receivers at reference locations providing corrections and relative positioning data for remote receivers. This method has been widely used in surveying, geodetic control, and plate tectonic studies where high accuracy positioning was needed.

2.2 GPS Accuracy

GPS was developed to support the broadest possible spectrum of users with accuracy levels to satisfy their various needs. There are two basic categories of GPS services:

PPS (precise positioning service) is a highly accurate positioning, velocity and timing service, which is made available only to authorized users (generally this means the military).

SPS (standard positioning service)-A less-accurate positioning and timing service, which is available to all GPS users, including civilian aviation.

Functionally, the PPS and SPS are nearly identical. The essential difference between them is the attainable accuracy.

2.2.1 Precise Positioning Service (PPS)

Authorized users with cryptographic equipment and keys and specially equipped receivers use the Precise Positioning System. U. S. and Allied military, certain U. S. Government agencies, and selected civil users specifically approved by the U. S. Government can use the PPS.

PPS Predictable Accuracy:

22 meter Horizontal accuracy

27.7 meter vertical accuracy

100 nanosecond time accuracy

PPS access is controlled by two features using cryptographic techniques:

- 1). The SA (selective availability) feature is used to reduce the GPS position, velocity, and time accuracy by inserting controlled errors into the satellite signals.

In May 1, 2000, the president of United States announced the US would stop the intentional degradation of the Global Positioning System (GPS) signals available to the public beginning at midnight of that day. This means the degradation feature Selective Availability (SA) was terminated. The civilian users of GPS are now able to pinpoint locations up to ten times more accurately than they previously did.

2). An A-S (anti-spoofing) feature is invoked at random times, without warning, to prevent potential spoofing (hostile imitation) of PPS signals. A-S alters the P-code cryptographically into a code denoted as the Y-code (the C/A-code remains unaffected).

Encryption keys and techniques are provided to PPS users, which allows removal of the **SA** and **A-S** effects and thereby attain the maximum available accuracy of GPS. PPS capable receivers not having the proper encryption keys installed suffer from accuracy degradations and are unable to track the Y-code.

PPS receivers can use either the P(Y)-code or C/A-code or both. Maximum accuracy is attained when using the P(Y)-code on L1 and L2 frequencies. The difference in propagation delay between the two frequencies is used to calculate ionospheric corrections (time delays). C/A-only receivers use an ionospheric model to calculate ionospheric corrections, as the C/A-code is broadcast only on L1-making dual

frequency delay measurements impossible. This will result in somewhat less positioning accuracy than the dual-frequency P(Y)-code receivers.

2.2.2 Standard Positioning Service (SPS)

Civil users worldwide use the SPS without charge or restrictions. Most receivers are capable of receiving and using the SPS signal. The SPS accuracy is intentionally degraded by the DOD using of Selective Availability. SPS is intended primarily for civilian purposes, although it has many peacetime military uses, and is also the service that general aviation use (Fig.2.5).

SPS Predictable Accuracy:

- 100 meter horizontal accuracy

- 156 meter vertical accuracy

- 340 nanoseconds time accuracy

When SA was turned off in May 2000, GPS measurement of position significantly improved. SA, or Selective Availability, was the intentional degradation of the SPS, or Standard Positioning Service, navigation data in order to deprive a military adversary of real-time more precise positioning. The Fig. 2.6 compares a day with and without SA using the same GPS receiver [148].

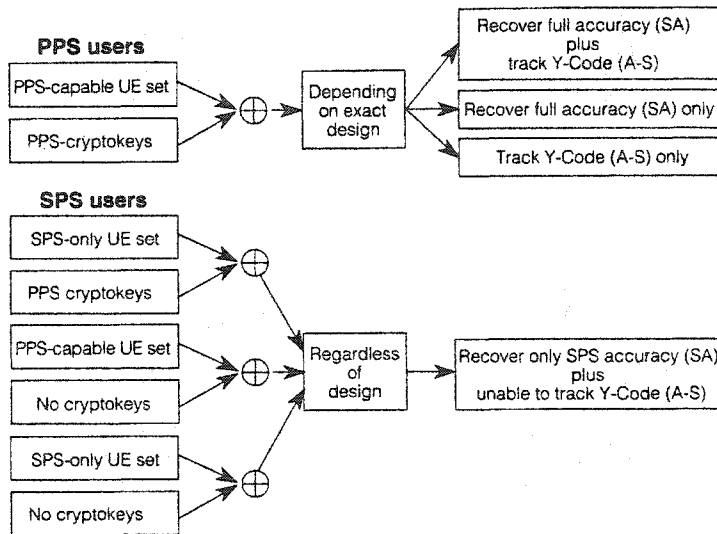


Fig. 2.5 PPS and SPS GPS Receiver Capabilities / Limitations

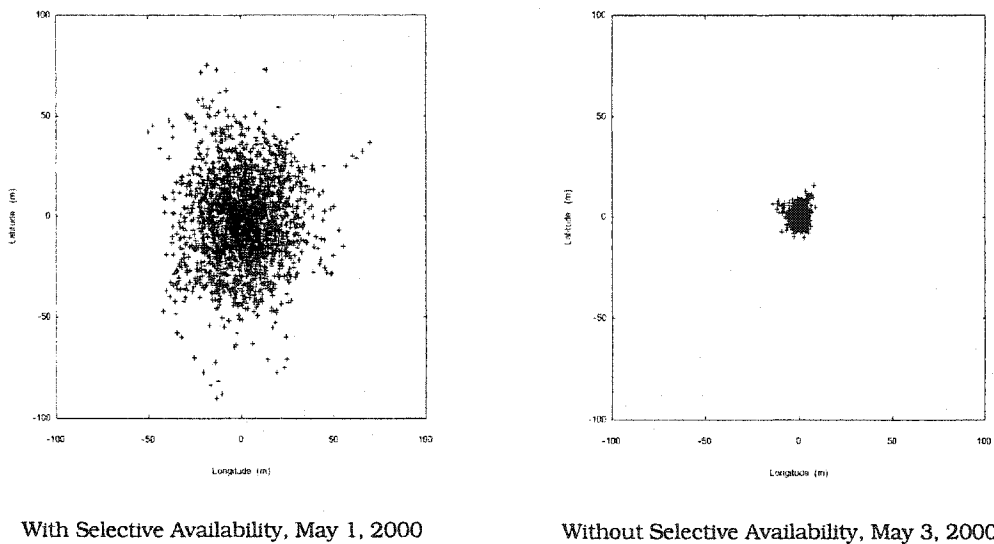


Fig.2.6 Comparison of Positions With and Without Selective Availability

The Figure 2.6 shows that SA causes 95% of the points to fall within a radius of 45.0 meters. Without SA, 95% of the points fall within a radius of 6.3 meters.

2.3 GPS Satellite Signals

The SVs transmit two microwave carrier signals. The L1 frequency (1575.42 MHz) carries the navigation message and the SPS code signals. The L2 frequency (1227.60 MHz) is used to measure the ionospheric delay by PPS equipped receivers [2].

The C/A Code (Coarse Acquisition) modulates the L1 carrier phase. The C/A code is a repeating 1 MHz Pseudo Random Noise (PRN) Code. This noise-like code modulates the L1 carrier signal, "spreading" the spectrum over a 1 MHz bandwidth. The C/A code repeats every 1023 bits (one millisecond). There is a different C/A code PRN for each SV. GPS satellites are often identified by their PRN number, the unique identifier for each pseudo-random-noise code. The C/A code that modulates the L1 carrier is the basis for the civil SPS.

The P-Code (Precise) modulates both the L1 and L2 carrier phases. The P-Code is a very long (seven days) 10 MHz PRN code. In the Anti-Spoofing (AS) mode of operation, the P-Code is encrypted into the Y-Code. The encrypted Y-Code requires a classified AS Module for

each receiver channel and is only for authorized users with cryptographic keys. The P (Y)-Code is the basis for the PPS.

The Navigation Message also modulates the L1-C/A code signal. The Navigation Message is a 50 Hz signal consisting of data bits that describe the GPS satellite orbits, clock corrections, and other system parameters.

2.4 Pseudo-Range Navigation

The position of the receiver is where the pseudo-ranges from a set of SVs intersects. Position is determined from multiple pseudorange measurements at a single measurement epoch. The pseudo range measurements are used together with SV position estimates based on the precise orbital elements (the ephemeris data) sent by each SV. The orbital data allow the receiver to compute the SV positions in three dimensions at the instant they send their respective signals.

Four satellites (normal navigation) can be used to determine three position dimensions and time. Position dimensions are computed by the receiver in **Earth-Centered, Earth-Fixed X, Y, Z (ECEF XYZ)** coordinates (Fig.2.7).

Time is used to correct the offset in the receiver clock, allowing the use of an inexpensive receiver clock. SV Position in XYZ is computed from four SV pseudo-ranges and the clock correction and ephemeris data. Receiver position is computed from the SV positions, the measured pseudo-ranges (corrected for SV clock offsets, ionospheric delays, and relativistic effects), and a receiver position estimate (usually the last computed receiver position).

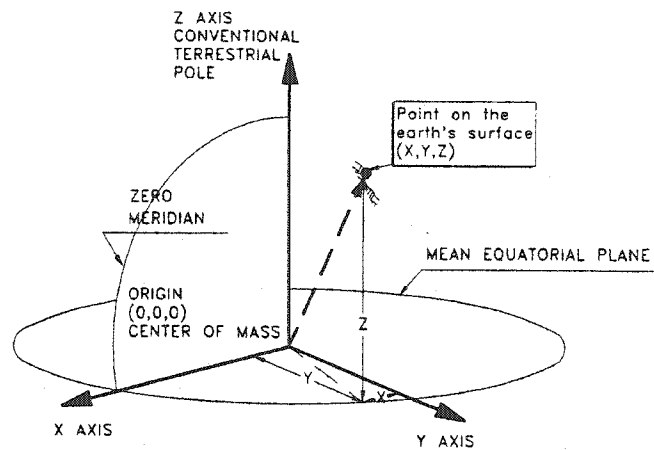


Fig.2.7 ECEF X, Y, and Z

If the clocks on the satellites and the receivers are synchronized, and all other conditions are perfect, then three satellites would be sufficient to determine the position of the receiver (Fig. 2.8) by Eq. 2.1:

$$\begin{aligned}
 R_1^2 &= (x_1 - x)^2 + (y_1 - y)^2 + (z_1 - z)^2 \\
 R_2^2 &= (x_2 - x)^2 + (y_2 - y)^2 + (z_2 - z)^2 \\
 R_3^2 &= (x_3 - x)^2 + (y_3 - y)^2 + (z_3 - z)^2
 \end{aligned}
 \tag{Eq. 2.1}$$

Where R_1, R_2, R_3 are the distances from the receiver to the satellites, $x_1, y_1, \dots, x_3, y_3, z_3$ are the coordinates of the reference satellites, and x, y, z are the coordinates of the receiver.

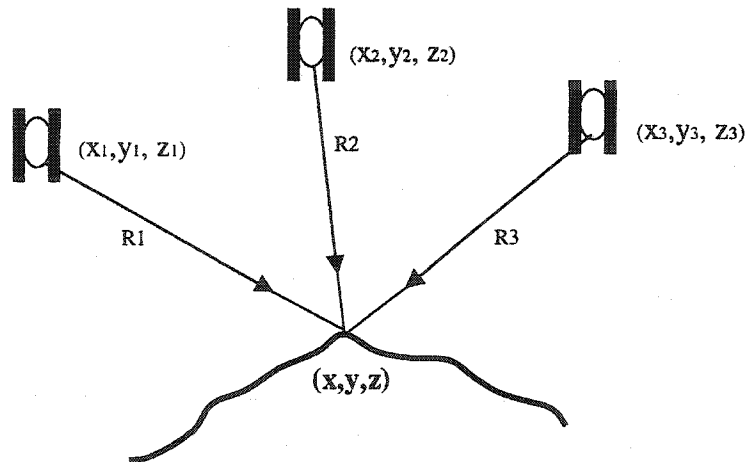


Fig. 2.8 Theoretically Three Satellites Could Position a Receiver

The three unknowns in Eq.2.1 above should have a unique solution. However, due to the concern for economy, a clock on a receiver is not as accurate as those on the satellites. The timing discrepancy between the clock on the receiver and the clocks on the satellites is called the clock bias, or clock offset. The clock bias causes the same amount of error on all distances it measures to the satellites.

Now there are four unknowns, x , y , z , and Δr , the distance error caused by clock bias. With four satellites, the coordinates of the receiver (x, y, z) , and the distance error Δr , as shown in Fig. 2.9, can be solved from the four equations in Eq.2.2.

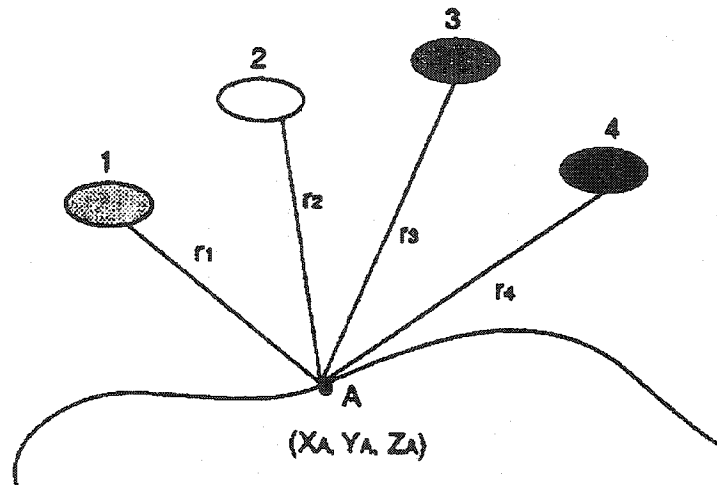


Fig. 2.9 Four Satellites Solution

$$\begin{aligned}
 (r_1 + \Delta r)^2 &= (x_1 - x)^2 + (y_1 - y)^2 + (z_1 - z)^2 \\
 (r_2 + \Delta r)^2 &= (x_2 - x)^2 + (y_2 - y)^2 + (z_2 - z)^2 \\
 (r_3 + \Delta r)^2 &= (x_3 - x)^2 + (y_3 - y)^2 + (z_3 - z)^2 \\
 (r_4 + \Delta r)^2 &= (x_4 - x)^2 + (y_4 - y)^2 + (z_4 - z)^2
 \end{aligned}$$

Eq.2.2

In practice, a receiver needs to measure (lock) at least four satellites. When there are more than four satellites locked, the least squares method is employed to get the best-estimated solutions.

Three satellites could be used to determine three position dimensions with a perfect receiver clock. In practice this is rarely possible and three SVs are used to compute a two-dimensional, horizontal fix (in latitude and longitude) given an assumed height. This is often possible at sea or in altimeter equipped aircraft. Five or more satellites can provide position, time and redundancy. More SVs can provide extra position fix certainty and allow detection of out-of-tolerance signals under certain circumstances.

2.5 Receiver Position, Velocity, and Time

Position in XYZ is converted within the receiver to geodetic latitude, longitude and height above the ellipsoid. Latitude and longitude are usually provided in the geodetic datum on which GPS is based (WGS-84). Receivers can often be set to convert to other user-required datums. Position offsets of hundreds of meters can result from using the wrong datum.

Velocity is computed from change in position over time, the SV Doppler frequencies, or both.

2.6 SV Time, GPS Time, and UTC

SV Time is the time maintained by each satellite. Each SV contains four atomic clocks (two cesium and two rubidium). SV clocks are monitored by ground control stations and occasionally reset to maintain time to within one-millisecond of GPS time. Clock correction data bits reflect the offset of each SV from GPS time.

SV Time is set in the receiver from the GPS signals, which is converted to GPS Time in the receiver. Data bit subframes occur every six seconds and contain bits that resolve the Time of Week to within six seconds. The 50 Hz data bit stream is aligned with the C/A code transitions so that the arrival time of a data bit edge (on a 20 millisecond interval) resolves the pseudo-range to the nearest millisecond. Approximate range to the SV resolves the twenty millisecond ambiguity, and the C/A code measurement represents time to fractional milliseconds. Multiple SVs and a navigation solution (or a known position for a timing receiver) permit SV Time to be set to an

accuracy limited by the position error and the pseudo-range error for each SV.

GPS Time is a "paper clock" ensemble of the Master Control Clock and the SV clocks. GPS Time is measured in weeks and seconds from 24:00:00, January 5, 1980 and is steered to within one microsecond of UTC. GPS Time has no leap seconds and is ahead of UTC by several seconds. Time in Universal Coordinated Time (UTC) is computed from GPS Time using the UTC correction parameters sent as part of the navigation data bits.

At the transition between 23:59:59 UTC on June 30, 1997 and 00:00:00 UTC on July 1, 1997, UTC was retarded by one-second. GPS Time is now ahead of UTC by thirteen seconds.

2.7 Carrier Phase Tracking (Surveying)

Carrier-phase tracking of GPS signals has resulted in a revolution in land surveying. A line of sight along the ground is no longer necessary for precise positioning. Positions can be measured up to 30 km from a reference point without intermediate points. This use of GPS requires specially equipped carrier tracking receivers.

The L1 and/or L2 carrier signals are used in carrier phase surveying. L1 carrier cycles have a wavelength of about 19 centimeters. If tracked and measured, these carrier signals can provide measurements with relative accuracies of millimeters under special circumstances.

Tracking carrier phase signals provides no time transmission information. The carrier signals, while modulated with time tagged binary codes, carry no time-tags that distinguish one cycle from another. The measurements used in carrier phase tracking are differences in carrier phase cycles and fractions of cycles over time. At least two receivers track carrier signals at the same time. Ionospheric delay differences at the two receivers must be small enough to insure that carrier phase cycles are properly accounted for. This usually requires that the two receivers be within about 30 km of each other.

Carrier phase is tracked at both receivers and the changes in tracked phase are recorded over time in both receivers.

Phase difference changes in the two receivers are reduced, using software, to differences in three position dimensions between the reference station and the remote receiver. High accuracy range difference measurements with sub-centimeter accuracy are possible.

Problems result from the difficulty of tracking carrier signals when there is noise or while the receiver moves.

2.8 Accuracy of Position and Time

The accuracy of receiver/processor position and time solutions is determined by two important factors:

- Error in the measurement of pseudoranges from each satellite being tracked.
- Satellite-to-user geometry.

Knowing these factors for a specific GPS receiver at a particular point in time and space is important as it allows an understanding of the limitations of GPS and enables forecasting of the receiver's position and time accuracy. The issue of GPS accuracy can be complex. There is rarely enough room to show the necessary detail for a complete picture of GPS accuracy. An ideal description of GPS accuracy will have reference to several factors. The most common factors that should be included in a complete description of accuracy include the following:

- Required occupation time
- Type of data recorded (phase or pseudorange)
- Type of processing (phase or pseudorange)

- Environmental conditions
- Maximum allowable PDOP
- Minimum allowable signal strength
- Maximum allowable distance between base and rover receivers
- Horizontal accuracy versus vertical accuracy

2.9 Accuracy Limiting Factors

The error in the receiver's measurement of the pseudorange from each satellite is called the user equivalent range error (UERE) and is the product of:

- Stability of the particular satellite's clock, including SA
- Predictability of the satellite's orbit -- ephemeris data
- Errors in the satellite-broadcast NAV-msgs
- Precision of the GPS receiver PRN sequence tracking design
- Errors in the processor's calculation of the ionospheric model
- Errors in the corrections caused by tropospheric effects
- Errors caused by reflected signals entering the receiver antenna, multipath.
- Errors in the receiver's measurement of range caused by thermal noise, software accuracy, and inter-channel biases

2.10 GPS Error Sources

A standard GPS receiver can measure its coordinates within an accuracy of 30 to 100 meters, depending on the length of the observation and the design of the receiver. This accuracy is good enough for some applications but not for control networks. There are several error sources involved in GPS observations.

GPS errors are a combination of noise, bias, and blunders. Noise errors are the combined effect of PRN code noise (around 1 meter) and noise within the receiver noise (around 1 meter). Bias errors result from Selective Availability and other factors.

2.10.1 Selective Availability (SA)

SA is the intentional degradation of the GPS signals by a time varying bias. SA is controlled by the DoD to limit accuracy for non-U.S. Military and government users. The potential accuracy of the C/A code of around 30 meters is reduced to 100 meters (two standard deviations).

The SA bias on each satellite signal is different, and so the resulting position solution is a function of the combined SA bias from each SV used in the navigation solution. Because SA is a changing bias with low frequency terms in excess of a few hours, position solutions or individual SV pseudo-ranges cannot be effectively averaged over periods shorter than a few hours. Differential corrections must be updated at a rate less than the correlation time of SA (and other bias errors).

2.10.2 Other Bias Error Sources

- Ephemeris data errors: 1 meter
- SV clock errors uncorrected by Control Segment can result in one meter errors. Basically, the satellites and receivers both need very good clocks to do their job. The smallest error can throw off the "range measurement" from the receiver to the satellite by many 10's, 100's or even 1000's of metres. For example a 10 nanosecond (0.00000001 sec) error would cause a 3-metre error in the range.
- Tropospheric delays: 1 meter.

The troposphere is the lower part (ground level from 8 to 13 km) of the atmosphere that experiences the changes in temperature,

pressure, and humidity associated with weather changes.

Complex models of tropospheric delay require estimates or measurements of these parameters.

- Unmodeled ionosphere delays: 10 meters.

The ionosphere is the layer of the atmosphere from 50 to 500 km that consists of ionized air. The transmitted model can only remove about half of the possible 70 ns of delay leaving a ten meter unmodeled residual.

- Multipath: 0.5 meters.

Multipath is caused by reflected signals from surfaces near the receiver that can either interfere with or be mistaken for the signal that follows the straight line path from the satellite.

Multipath is difficult to detect and sometime hard to avoid.

- Blunders can result in errors of hundred of kilometers.
- Control segment mistakes due to computer or human error can cause errors from one meter to hundreds of kilometers.
- User mistakes, including incorrect geodetic datum selection, can cause errors from 1 to hundreds of meters.
- Receiver errors from software or hardware failures can cause blunder errors of any size.

- Noise and bias errors combine, resulting in typical ranging errors of around fifteen meters for each satellite used in the position solution.
-

2.11 Geometric Dilution of Precision (GDOP) and Visibility

The UERE depends, in part, on the quality of the broadcasted satellite signals and will vary between satellites and from time to time. It also depends on receiver/processor designs and can therefore vary from equipment to equipment.

GDOP is a very important factor in determining GPS accuracy. GDOP is a geometric quantity depending upon the relative positions of the user and the selected satellites. High values of DOP cause small range measurement errors to become large position errors. Therefore, the four satellites used by the receiver/processor to determine its position, velocity and time (PVT) should be selected to minimize high-DOP values. The limiting factor is independent of the quality of the broadcast satellite signals or the type of GPS receiver (providing the same four satellites are selected). DOP is, basically, an amplification

factor that multiplies the UEREs and increases the receiver/processor's units position/time solution errors.

GPS ranging errors are magnified by the range vector differences between the receiver and the SVs. The volume of the shape described by the unit-vectors from the receiver to the SVs used in a position fix is inversely proportional to GDOP.

Poor GDOP, a large value representing a small unit vector-volume, results when angles from receiver to the set of SVs used are similar.

Good GDOP, a small value representing a large unit-vector-volume, results when angles from receiver to SVs are different.

GDOP is computed from the geometric relationships between the receiver position and the positions of the satellites the receiver is using for navigation. For planning purposes GDOP is often computed from Almanacs and an estimated position. Estimated GDOP does not take into account obstacles that block the line-of-sight from the position to the satellites. Estimated GDOP may not be realizable in the field.

Good Computed GDOP and Bad Visibility Equals Poor GDOP. GDOP terms are usually computed using parameters from the navigation solution process.

In general, ranging errors from the SV signals are multiplied by the appropriate GDOP term to estimate the resulting position or time error. Various GDOP terms can be computed from the navigation covariance matrix. ECEF XYZ DOP terms can be rotated into a North-East Down (NED) system to produce local horizontal and vertical DOP terms.

GDOP Components

PDOP = Position Dilution of Precision (3-D), sometimes the Spherical DOP.

HDOP = Horizontal Dilution of Precision (Latitude, Longitude).

VDOP = Vertical Dilution of Precision (Height).

TDOP = Time Dilution of Precision (Time).

While each of these GDOP terms can be individually computed, they are formed from covariances and so are not independent of each other. A high TDOP (time dilution of precision), for example, will cause receiver clock errors which will eventually result in increased position errors.

2.12 Differential GPS (DGPS) Techniques

The idea behind all differential positioning is to correct bias errors at one location with measured bias errors at a known position. A

reference receiver, or base station, computes corrections for each satellite signal. Because individual pseudo-ranges must be corrected prior to the formation of a navigation solution, DGPS implementations require software in the reference receiver that can track all SVs in view and form individual pseudo-range corrections for each SV. These corrections are passed to the remote or rover receiver which must be capable of applying these individual pseudo-range corrections to each SV used in the navigation solution. Applying a simple position correction from the reference receiver to the remote receiver has limited effect at useful ranges because both receivers would have to be using the same set of SVs in their navigation solutions and have identical GDOP terms (not possible at different locations) to be identically affected by bias errors.

2.12.1 Differential Code GPS (Navigation)

Differential corrections may be used in real-time or later, with post-processing techniques. Real-time corrections can be transmitted by radio link. The U. S. Coast Guard maintains a network of differential monitors and transmits DGPS corrections over radio beacons covering much of the U. S. coastline. DGPS corrections are often transmitted in

a standard format specified by the Radio Technical Commission Marine (RTCM).

Corrections can be recorded for post processing. Many public and private agencies record DGPS corrections for distribution by electronic means. Private DGPS services use leased FM sub-carrier broadcasts, satellite links, or private radio-beacons for real-time applications. Today there are networks of land-based radio beacons around the world that constantly broadcast signals to receivers that enable them to differentially correct, or enhance the accuracy of, the data they receive from standard GPS.

To remove Selective Availability (and other bias errors), differential corrections should be computed at the reference station and applied at the remote receiver at an update rate that is less than the correlation time of SA. Suggested DGPS update rates are usually less than twenty seconds.

DGPS removes common-mode errors, those errors common to both the reference and remote receivers (not multipath or receiver noise). Errors are more often common when receivers are close together (less than 100 km). Differential position accuracies of 1-10 meters are possible with DGPS based on C/A code SPS signals.

2.12.2 Differential Carrier GPS (Survey)

All carrier-phase tracking is differential, requiring both reference and remote receiver tracking carrier phases at the same time. To correctly estimate the number of carrier wavelengths at the reference and remote receivers, they must be close enough to insure that the ionospheric delay difference is less than a carrier wavelength. This usually means that carrier-phase GPS measurements must be taken with a remote and reference station within about 30 kilometers of each other or longer if the DGPS software is designed by Kalman Filter method and an appropriate model has been selected.

Special software is required to process carrier-phase differential measurements. Newer techniques such as Real-Time-Kinematic (RTK) processing allow for centimeter relative positioning with a moving remote receiver.

CHAPTER 3 HIGH ACCURACY REFERENCE NETWORK (HARN)

3.1. Introduction

It is very important to establish a national geodetic network for mapping, engineering surveying, planning, and many other surveying purposes. To address these changing requirements for positional information, National Geodetic Survey NGS began 15 strategic goals in 1992. Formally adopted in March 1994 (NGS 1994), this plan describes the steps necessary to accomplish the transition from National Reference System (NRS) to the National Spatial Reference System (NSRS). The 1994 plan represented a fundamental change by NGS to understand the needs of geospatial data users, and to restructure its surveying operations, research and development activities, and data distribution, some of which have changed only slightly in over 180 years. These changes include developments of:

- Federal Base Network (FBN), Cooperative Base Network (CBN), User Densification Network – High Accuracy Reference Network (HARN) (fig.3.1)
- Continuously Operating Reference Stations (CORS)
- Accurate determination of GPS Orbits
- Method and determination of High Accuracy Geoid
- Improved data access.

In order to offer all government agencies and private users a new high accuracy National Spatial Reference System for GIS, and Transportation Engineering applications, NGS started a High Accuracy Reference Network (HARN) project in Tennessee in 1990 and completed it in 1997 in Indiana for all 50 States. HARN project not only offers a high reference network over for 50 States, it also brings together the many spatial reference points in all 50 states of the U.S.

NGS, in cooperation with Iowa Department of Transportation and Iowa State University, completed Iowa HARN project in 1996 (Fig.3.1). Data collection for the Iowa State HARN project was completed by the end of March 1997 and all GPS data processing for this research project was completed by Iowa State University by the end of April 1997.

There are two steps in establishing HARN:

- Establishing A-order points by DGPS using CORS as reference stations
- Establishing B-order points by DGPS using A-order as reference stations

3.2. Project Definition

The Iowa HARN will be defined by the requirements of surveying purposes and engineering applications, and resources of NGS, Iowa DOT. The figure 3.1 shows Iowa HARN network, which consists of 181

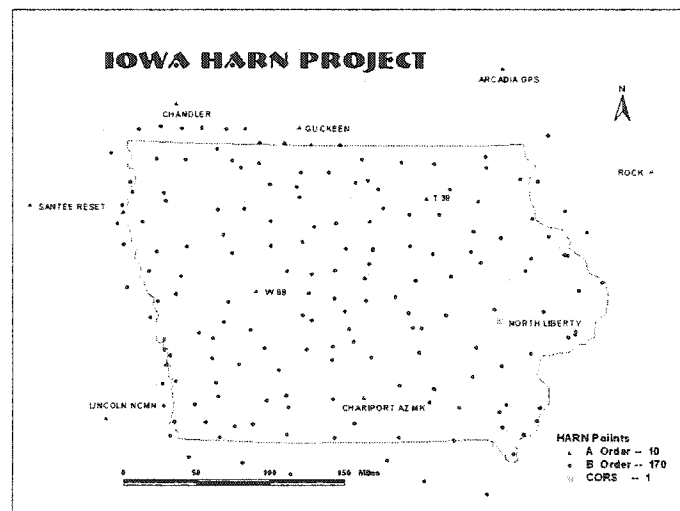


Fig.3.1 Iowa HARN Project

stations. In order to satisfy Federal Base Network (FBN) and NGS requirements, the GPS network will usually best perform an A-order survey. The A-order survey establishes stations at roughly 300 km spacing and is designed to achieve one part of 10,000,000 horizontal accuracy, and requires 3 days of observations at each site. In Iowa A-order stations consist of four monumented points; one of them is a Continuously Operating Reference Station (**CORS**).

Therefore, an A-order survey is required first. It must be completed before all B-order points are observed. In Iowa It was completed by 1996. Based on these A-order points and A-order points of other states around Iowa, the NGS established a 1 x 1 degree HARN in Iowa as part of the nationwide HARN. This required a GPS survey of approximately 141 B-order stations within Iowa. The B-order survey included the establishment of HARN stations at approximately 17 airports, to partially satisfy the FAA requirement. These HARN stations are labeled Primary Airport Control Stations (**PACS**) by Federal Aviation Administration (**FAA**).

The High Accuracy Reference Network (HARN) is a three dimensional control network used by NGS to upgrade the National Geodetic Reference System (NGRS). HARNs are designed to provide a nation-wide coverage of B-order accuracy control points. From these

control points, a denser geodetic control network can be developed for further applications. Iowa HARN, will serve as a high accuracy basic reference control network for Iowa agriculture, hydrology, high way construction, GIS, and many other applications.

To offer a high accuracy reference network for Iowa, there are 181 points in Iowa HARN project. The control area of each point can be seen in (Fig. 3.2). There are 142 stations (A_order and B_order) in Iowa. The mean area of each point covered is 396.04 square miles. The minimum area of a point covered is 67.97 square miles and the maximum area of a point covered is 767.36 square miles.

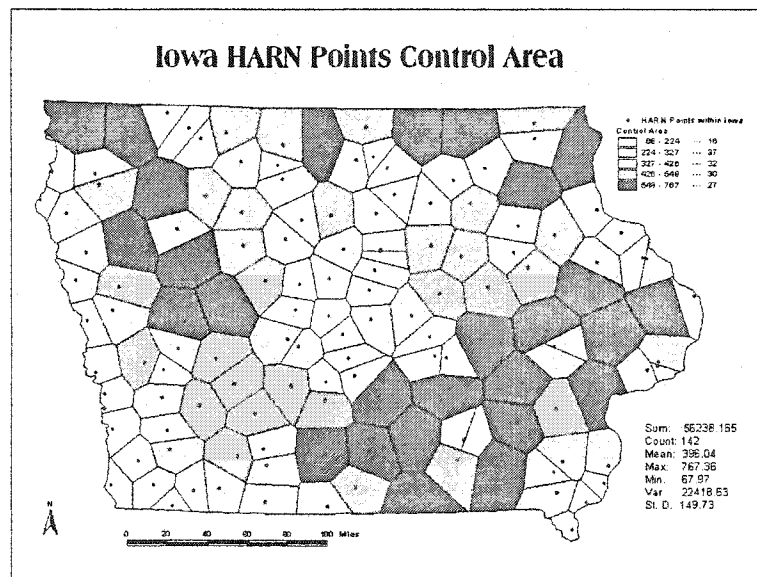


Fig. 3.2 Iowa HARN Points Control Areas

3.3. Continuously Operating Reference Stations (CORS) Network

3.3.1 Introduction

Establishment of a national network of GPS CORS station (Fig.3.3) will make both pseudo-range and carrier-phase measurements accessible to the public for use in a wide range of geodetic, cadastral,

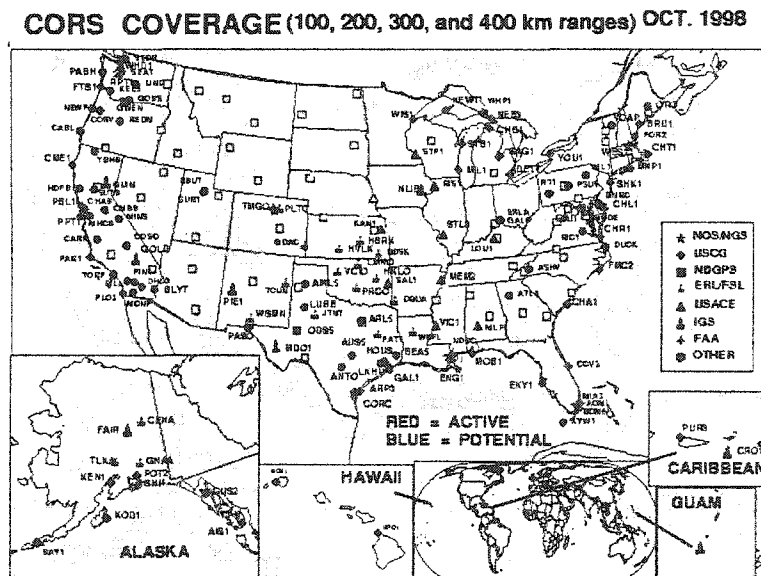


Fig. 3.3 GPS CORS Stations

and engineering surveying applications, GIS mapping and charting, as well as air, land, and sea navigation.

Each CORS will consist of:

- at least one permanent high accurate GPS receiver, which collects data 24 hours each day
- a computer to process and store the data
- a telecommunications support system to publish data online and allow user access

CORS is correlated with a FBN/CBN station located in a secured environment such as an airport, Coast Guard station, or other protected location. Data from these sites supported by the Coast Guard and FAA, will provide position and elevation corrections for real-time pseudo-range observations to improve accuracies for a variety of applications from the sub-meter level up to 200 kilometers away from the station.

The GPS CORS network coordinated by the National Geodetic Survey (NGS), National Oceanic and Atmospheric Administration (NOAA), is a group of GPS reference stations which provide code range and carrier phase data to users in support of post-processing applications. Government, academic, commercial and private users

will be supported in performing after-the-fact positioning of fixed points and moving platforms. Ultimately, the CORS network is expected to consist of nearly 300 stations located nationwide. All current sites will be operational at all times or will be kept as part of the final network configuration. Also all GPS data, positional information, utility software, or ancillary information which have been posted are perfect and without error; and the GPS data is being recorded at a 30, 15, 10, 5, and 1 second sampling rate in the Receiver INdependent EXchange (RINEX 2) format, version 2.0. The data sets are available for 31 days and can be retrieved over the INTERNET. After that period, the data are archived on CD ROM and will be available by special request.

3.3.2 CORS Station Operators

The following organizations (Table 3.1) provide data from their stations for distribution [167].

Table 3.1 National CORS Station Operators

Organization	# Sites
NOAA (FSL/NGS/CIGNET)	60
U.S. Coast Guard (DGPS)	49
U.S. Department of Transportation (NDGPS)	20
Pacific Northwest Geodetic Array (PANGA)	18
Michigan Department of Transportation (MDOT)	16
Jet Propulsion Laboratory (JPL/NASA)	14
North Carolina Geodetic Survey	14
Florida Department of Transportation	13
Texas Department of Transportation	12
U.S. Army Corps of Engineers (DGPS)	11
Basin and Range Geodetic Network (BARGN)	10

Table 3.1 (Continued)

Ohio Department of Transportation	10
Bay Area Regional Deformation Array, CA (BARD)	8
Oklahoma Department of Transportation	8
Bayonet Network	7
Southern California Integrated GPS Array (SCIGN)	7
Federal Aviation Administration (FAA)	6
BP Exploration	5
Pennsylvania Department of Transportation	5
Louisiana State University	4
University of Utah / UNAVCO	4
Harris Galveston Coastal Subsidence District, TX	3
Pacific GPS	3
Instituto Geografico Nacional, Guatemala	2
Louisiana State University	2
South Carolina Geodetic Survey	2
American Samoa Department of Commerce, AS	1
Carbon County, UT	1
Central Washington University, WA	1
City of Cincinnati, OH	1
City of High Point, NC	1
City of Kingman, AZ	1
City of Scottsdale, AZ	1
City of Tucson, AZ	1
Columbia County, GA	1
Commonwealth of the Northern Mariana Islands, TQ	1
Condor Earth Technologies Inc, CA	1
Delaware Department of Parks and Recreation	1
Greenville Technical Institute, SC	1
Hagerstown Community College, MD	1
Idaho Department of Transportation	1
Indiana University	1
Lake County Division of Transportation, IL	1
Lamont Earth Observatory, NY	1
Las Vegas Valley Water District	1
Lawrence Livermore National Laboratory, CA	1
Minnesota Department of Transportation	1
Missoula County, MT	1
NCAD Corporation, KY	1
New Jersey Institute of Technology	1
Northampton Department of Public Works	1
North Dakota Department of Transportation	1
Ohio University	1
Oklahoma Department of Transportation	1
Paul Smith College, NY	1
Riverside County Flood District, CA	1
Salt Lake County, UT	1
Salt River Project, AZ	1
Southeastern Polytechnic University	1
The Surveyors Exchange, AK	1
University of Illinois	1
University of Virginia-Fan Mountain, VA	1
Vermont Agency of Transportation	1
Virginia Department of Transportation	1

Total

351

3.3.3 Availability

RINEX format data is available from the following stations (Table 3.2)

[167].

Table 3.2 CORS station data availability

SITE NAME	ST	MON. NAME	ORGANIZATION	DAY NUMBER							
				062	063	064	065	066	067	068	
*Acushnet 1	MA	ACU1	USCG	B50	B50	B50	B50	B50	B50	H48	
Ada	OK	OKAD	OKDOT	B51	B50	B51	B51	B51	B51	B49	
Addicks 1795	TX	ADKS	HGCSD	D2	D2	0	D2	D2	D2	0	
Adrian	MI	ADRI	MDOT	B26	B26	B26	B25	B26	B26	H24	
*Albuquerque UNM	UNM	ABQ1	USCG	B50	B50	B50	B50	B50	B50	H48	
*Albuquerque UNM	UNM	ABQ2	USCG	0	0	0	0	B6	0	0	
*Albuquerque WNM	WNM	ZAB1	FAA	B50	B50	B50	B50	B50	B50	H48	
*Albuquerque WNM	WNM	ZAB2	FAA	0	0	0	0	0	0	0	
Alpena	MI	NOR3	MDOT	B24	B26	B25	B25	B25	B24	H24	
Alternate MasCO	AMC2	JPL	JPL	D2	D2	D2	D2	D2	D2	0	
Amarillo	TX	AML5	TXDOT	0	0	0	0	0	0	0	
American SamoAS	ASPA	ASDC	ASDC	B26	B26	B27	B26	B27	B26	B26	
*Anchorage WAAAK	ZAN1	FAA	FAA	B50	B50	B50	B50	B50	B50	H48	
*Annapolis 1	MD	ANP1	USCG	B50	B48	B50	B50	B50	B50	H48	
*Annette IslanAK	AIS1	USCG	USCG	B50	B50	B50	B50	B50	B50	H48	
Antlers	OK	OKAN	OKDOT	B51	B48	B51	B51	B51	B51	B49	
Apalachicola	FL	MRKB	FDOT	B50	B50	B48	B50	B50	B48	H48	
*Appleton 1	WA	GWEN	USCG	B50	B50	B50	B50	B42	B50	H18	
*Aransas Pass	TX	ARP3	USCG	B50	B50	B50	B50	B50	B50	H48	
Arlington RRPTX	ARL5	TXDOT	TXDOT	0	0	0	0	0	0	0	
Asheville	NC	ASHV	NCGS	B50	B50	B50	B40	B50	B50	H48	
Atlanta	GA	ATL1	SPSU	0	0	0	0	0	0	0	
*Atlanta WAAS	GA	ZTL4	FAA	B50	B50	B50	B50	B50	B50	H48	
Atlantic OceaFL	AOML	NGS	NGS	D3	D3	D3	D3	D3	D3	D3	
Auburn Hills	MI	METR	MDOT	B25	B26	B25	B25	B25	B24	H24	
Austin RRP	TX	AUS5	TXDOT	0	0	0	0	0	0	0	
Aztec	NM	AZCN	FSL	Y51	Y51	Y51	+3	Y51	Y75	72	
Bar Harbor	ME	BARH	NGS	Y27	Y27	B19	Y7	Y27	Y27	Y28	
Bartlett	NH	BARN	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72	
Beaumont RRP	TX	BEA5	TXDOT	0	0	0	0	0	0	0	
*Billings 1	MT	BIL1	USCG	B50	B50	B50	B50	B50	B50	H48	
*Billings 2	MT	BIL2	USCG	0	0	0	0	0	0	0	
*Biorca IslandAK	BIS1	USCG	USCG	B50	B50	B50	B50	B50	B50	H48	
Bismark	ND	BSMK	NDDOT	B27	B27	B27	B27	B27	B27	B27	
Blacksburg	VA	BLKV	FSL	Y51	Y51	Y51	Y51	Y51	Y71	68	
Bloomfield	MO	BLMM	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72	
Blue River	WI	BLRW	FSL	Y43	Y51	Y51	Y23	Y51	Y75	72	
Blythe	CA	BLYT	SOPAC	D2	D2	D2	D2	D2	1	0	
Boothville VeLA	BVHS	LSU	LSU	D2	D2	D2	D2	0	0	0	
Boulder	CO	DSRC	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72	
Brighton	MI	BRIG	MDOT	B26	B26	B26	B25	B26	B25	H24	
*Brunswick 1	ME	BRU1	USCG	B50	B50	B50	B50	B50	B50	H48	
Buffalo	NY	BFNY	NGS	B49	B51	B51	B45	B38	B44	B28	
Burns Junction	OR	BURN	UWA	D2	D2	D2	D2	D2	D2	0	
Cadillac	MI	NOR1	MDOT	B24	B26	B25	B25	B25	B24	H24	
Cape Blanco	OR	CABL	CWU	D2	D2	D2	D2	D2	D2	0	
*Cape CanaveraFL	CCV3	USCG	USCG	0	0	0	0	0	0	0	
*Cape HinchinAK	CHI3	USCG	USCG	B50	B50	B50	B50	B46	B36	H48	

Table 3.2 (continued)

*Cape HinchinBAK	CHI4	USCG	B50	B50	B50	B50	B44	B24	H48
Cape Mears	OR	CHZZ	CWU	D2	D2	D2	D2	D2	0
*Cape MendocinCA	CME1	USCG	B50	B50	B50	B50	B50	B50	H48
Carbon CountyUT	PUC1	CCO	B26	B25	B27	B27	B26	B27	H24
Carrhill	CA	CARR	SOPAC	D2	D2	D2	0	1	0
Casper	WY	CASP	NGS	B51	B51	B51	B51	B51	B51
Castle Hayne	NC	CASL	NCGS	B50	B50	B50	B50	B50	B50
Central	AK	CENA	FSL	Y51	Y51	Y47	Y41	Y47	Y75
Chabot	CA	CHAB	USGS	D2	D2	D2	D2	0	0
Charles Pack	WA	CPXF	CWU	D2	D2	D2	D2	0	0
*Charleston 1	SC	CHA1	USCG	B50	B50	B50	B50	B50	H48
*Chatham 1	MA	CHT1	USCG	0	0	0	0	0	0
*Cheboygan 1	MI	CHB1	USCG	B50	B50	B50	B50	B50	H48
Chesapeake LiVA	COVX	FSL	B48	B50	B44	B48	B48	B50	H48
*Chico 1	CA	CHO1	USCG	B50	B50	B50	B50	B50	H48
*Chico 2	CA	CHO2	USCG	0	0	0	0	0	0
China Lake	CA	COSO	SOPAC	D2	D2	D2	D2	0	0
City of TucsoAZ	COT1	CofT	B48	B50	B34	B50	B50	B50	H48
*Clark 1	SD	CLK1	USCG	B50	B50	B49	B50	B50	H48
*Cold Bay 1	AK	BAY1	USCG	B44	B6	B48	B50	B50	B50
Columbia	SC	COLA	SCGS	B50	B50	B50	B50	B50	H48
Columbia CollCA	CMBB	UCB	D2	D2	D2	D2	D2	D3	0
Columbia CounGA	COGA	TRS	B50	B50	B50	B48	B50	B50	H48
Columbus	OH	COLB	OHDOT	B51	B49	B51	B51	B51	H48
Commonwealth	MP	CNMI	CNMI	0	0	0	0	0	0
Condor	CA	CNDR	CETI	D2	D2	D2	D2	D2	D2
Conover	NC	CONO	NCGS	B50	B50	B50	B50	B50	H48
Conway	MO	CNWM	FSL	Y50	Y51	Y51	Y51	Y51	Y75
Corbin	VA	CORB	NGS	B26	B24	B26	B26	B26	B14
Corpus ChristTX	CORC	TXDOT	0	0	0	0	0	0	0
Corvallis	OR	CORV	OSU	D2	D2	D2	D2	D2	0
Defiance	OH	DEFI	OHDOT	B51	B51	B51	B50	B50	B48
DeFuniak SpriFL	DFNK	FDOT	B50	B50	B48	B50	B48	B50	H48
Dequeen	AR	DQUA	FSL	Y51	Y51	Y47	Y51	Y51	Y75
*Detroit 1	MI	DET1	USCG	0	0	0	0	0	0
Dodson Butte	OR	DDSN	PANGA	D2	D2	D2	D2	D2	0
Dover	DE	DNRC	DEDP	Y51	Y51	Y51	Y51	Y51	Y75
Drill Site LiAK	DSL1	BP	B51	B41	B45	B47	B42	B42	H32
*Driver 1	VA	DRV1	USCG	B50	B50	B50	B50	B50	H48
*Driver 2	VA	DRV2	USCG	0	0	0	0	0	0
Durmid Hill	CA	DHLG	SOPAC	0	D2	D2	D2	D2	1
Dyer	NV	DYER	UNAVCO	D2	D2	D2	D2	D2	1
East Dock	AK	EDOC	BP	B49	B49	B49	B49	B46	B48
East Port EleME	EPRT	NGS	B26	B26	B26	B26	B26	B26	H22
Echo Canyon	SNV	ECHO	UNAVCO	D2	D2	D2	D2	D2	1
Egan	NV	EGAN	HSCfA	D2	D2	D2	D2	D2	1
EL Paso 3	TX	ELP3	TXDOT	0	0	0	0	0	0
*English Turn	LA	ENG1	USCG	0	0	0	0	0	0
Erlanger	KY	ERLA	NCAD	B50	B48	B50	B50	B50	B49
Escanaba	MI	SUP2	MDOT	B24	B26	B25	B24	B25	B24
Esteli	NU	ESTI	NGS	D1	D1	0	0	D1	D1
Fairbury	NE	FBYN	FSL	Y51	Y51	Y51	Y51	Y51	Y75
Fallon Naval	NV	NFLN	FSL	Y51	Y51	Y51	Y51	Y51	Y75
Fayetteville	NC	FAYR	NCGS	B50	B50	B50	B50	B50	H48
Ferno Mesa	AZ	FERN	UNAVCO	D2	D2	D2	D2	D2	1
*Flagstaff 1	AZ	FST1	USCG	B50	B48	B50	B50	B50	H48
Florence DarlSC	FDTC	SCGS	B27	B27	B27	B27	B27	B27	B27
*Fort Stevens	OR	FTS1	USCG	B50	B50	B50	B50	B50	H48
Fredonia	AZ	FRED	UNAVCO	D2	D2	D2	D2	D2	0
Freeport	OH	FREO	OHDOT	B50	B50	B50	B50	B48	B50
Gabbs	NV	GABB	UNAVCO	D2	D2	D2	D2	D2	1
Gainesville	FL	GNVL	FDOT	B50	B50	B48	B50	B50	H48

Table 3.2 (continued)

Gaithersburg MD	GAIT	NGS	B36	B16	B50	B50	B50	B50	B50	H48
Galbraith OH	GALB	CofC	B51	B51	B51	B51	B51	B51	B51	H48
*Galveston 1 TX	GAL1	USCG	B50	B48	B50	B50	B50	B50	B50	H48
*Galveston 2 TX	GAL2	USCG	0	0	0	0	0	0	0	0
Garlic NV	GARL	UNAVCO	D2	D2	D2	D2	D2	1	0	0
Gaston NC	GAST	NCGS	0	0	0	0	0	0	0	0
Gaylord MI	NOR2	MDOT	B24	B26	B25	B25	B25	B24	H24	
GGAO MD	GODE	JPL	D2	D2	D2	D2	D2	D2	0	
Gilmore Creek AK	FAIR	JPL	D2	D2	D2	D2	D2	D2	0	
Glennallen AK	GNAA	FSL	Y51	Y51	Y51	Y51	Y51	Y75	70	
Gloucester PoVA	GLPT	NGS	+3	+3	+3	+3	+3	+3	+4	
Goldendale ObWA	GOBS	CWU	D2	D2	D2	D2	D2	D2	0	
Goldstone 2 CA	GOL2	JPL	D2	D2	D2	D2	D2	D2	0	
Granada CO	GDAC	FSL	Y51	Y51	Y51	Y49	Y51	Y75	72	
Grand Marais MN	GDMA	NGS	D2	D2	D3	D2	D3	D3	0	
Grand Rapids MI	GRAR	MDOT	B25	B26	B26	B25	B26	B25	H24	
Greenville TeSC	GVLТ	GTI	B50	B46	B50	B50	B50	B38	0	
Guatemala GT	GUAT	NGS	B27	B27	B27	B27	B22	B27	H24	
Gustavus OH	GUST	OHDOT	B50	B49	B50	B50	B50	B50	H48	
*Gustavus 2 AK	GUS2	USCG	B50	B50	B50	B50	B50	B50	H48	
*Hagerstown 1 MD	HAG1	USCG	0	0	0	0	0	0	0	
*Hagerstown 2 MD	HAG2	USCG	B50	B50	B50	B50	B50	B50	H48	
Hagerstown CoMD	HTCC	HCC	B26	B26	B26	B26	B26	B26	H24	
Haleakala HI	MAUI	PGPSF	D2	D2	D2	D2	D2	0	0	
Hammond LA	HAMM	LSU	D2	D2	D3	D3	D3	D3	0	
*Hartsville 1 TN	HTV1	USCG	B50	B50	B50	B50	B50	B50	H48	
Haskell OK	HKLO	FSL	Y49	Y51	Y51	Y51	Y51	Y75	72	
Haviland KS	HVLK	FSL	Y51	Y51	Y49	Y51	Y51	Y75	72	
*Hawk Run 1 PA	HRN1	USCG	B50	B50	B50	B50	B50	B50	H48	
Haywood NC	HAYW	NCGS	0	0	0	0	0	0	0	
High Point NC	HIPT	CofHP	B50	B50	B50	B48	B50	B50	H48	
Hillsboro KS	HBRK	FSL	Y51	Y45	Y51	Y51	Y51	Y75	72	
Hillsborough NC	HILB	NCGS	B50	B50	B50	B50	B50	B50	H48	
Hilo Airport HI	HIL0	PGPSF	0	0	0	0	0	0	0	
Honolulu TideHI	HNLC	PGPSF	D2	D2	D2	D2	D2	0	0	
Hopland - BARCA	HOPB	UCB	D2	D2	D2	D2	D2	1	0	
Horn Point EnMD	HNPT	NGS	D2	D2	D2	D2	D2	D2	D3	
Houston RRP TX	HOUS	TXDOT	0	0	0	0	0	0	0	
*Hudson Falls NY	HDF1	USCG	B50	B50	B50	B50	B50	B50	H48	
HuehuetenangoGT	HUEH	NGS	0	0	0	0	0	0	0	
Idaho DOT ID	IDTD	NGS	D1	D1	0	B12	B51	B51	H28	
Ilopango AirpES	SSIA	NGS	0	0	0	0	D3	D3	0	
Indiana UniveIN	IUCO	INUN	B50	B50	B50	B50	B50	B50	H48	
Jackson MI	UNIV	MDOT	B24	B26	B25	B24	B25	B24	H24	
Jacksonville FL	JXVL	FDOT	B50	B50	B48	B50	B50	B50	H48	
*Jacksonville FL	ZJX1	FAA	B50	B50	B50	B50	B50	B50	H48	
Jayton TX	JTNT	FSL	Y45	Y51	Y51	Y51	Y51	Y75	72	
Kalamazoo MI	SOWR	MDOT	B24	B26	B25	B24	B25	B24	H24	
*Kansas City 1KS	KAN1	USCG	B50	B50	B50	B50	B50	B50	H48	
Kelso WA	KELS	UWA	D2	D2	D2	D2	D2	D2	0	
*Kenai 1 AK	KEN1	USCG	B50	B50	B50	B50	B50	B50	H48	
Key West FL	KWST	FDOT	B50	B50	B48	B50	B50	B48	H48	
*Key West 1 FL	KYW1	USCG	B50	B50	B50	B50	B50	B50	H48	
*Key West 2 FL	KYW2	USCG	0	0	0	0	0	0	0	
Kingman AZ	KING	KING	D3	D3	D3	D3	D3	D3	0	
Kingston JA	JAMA	NGS	D3	D3	0	D3	0	0	0	
Kissimmee FL	KSME	FDOT	B50	B50	B48	B50	B50	B50	H48	
KJUN LA	KJUN	LSU	D2	D2	D3	D3	D3	D3	0	
Kodiak AK	KODK	SOPAC	0	D2	D2	0	0	D2	0	
*Kodiak 1 AK	KOD1	USCG	B50	B50	B50	B50	B50	B50	H48	
Kokee Park GeHI	KOKB	JPL	D2	D2	D2	D2	0	D2	0	

Table 3.2 (continued)

*Kokole Point HI	KOK1	USCG	B50	B50	B50	B50	B50	B50	B50	H48
Kwajalein Atoll	KWJ1	JPL	0	0	0	0	0	0	0	0
L'Anse MI	SUP1	MDOT	B24	B26	B25	B24	B25	B24	H24	
Lake County DIL	LCDT	LCDT	B27	B16	B19	B14	B18	B8	H8	
Lake Houston TX	LKHU	HGCSD	D2	D2	D2	D2	D2	D2	0	
Lake Skinner CA	BILL	SOPAC	D2	D2	D2	D2	D2	1	0	
Lamont OK	LMNO	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72	
Lamont Earth NY	LAMT	LAMT	B27	B27	B27	B27	B27	B27	H24	
Lansing MI	LANS	MDOT	B25	B26	B25	B25	B26	B24	H24	
Laredo RRP TX	LARD	TXDOT	0	0	0	0	0	0	0	
Las Vegas Val NV	LVWD	LVVWD	B27	B27	B27	B22	B27	B26	H23	
Lathrop MO	LTHM	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72	
Lawton OK	OKLW	OKDOT	B50	B50	B50	B50	B50	B50	H48	
Lebanon OH	LEBA	OHDOT	B50	B49	B50	B49	B48	B48	H46	
*Level Island AK	LEV1	USCG	B50	B14	B50	B50	B50	B50	H48	
Lillington NC	LILL	NCGS	B50	B50	B50	B50	B50	B50	H48	
Lind Hall WA	LINH	CWU	D2	D2	D2	D2	D2	D2	0	
LLNL Site 300CA	S300	LLNL	B50	B50	B48	B50	B50	B50	H48	
*Louisville 1 KY	LOU1	USCG	B50	B50	B42	B46	B50	B50	H48	
Lubbock RRP TX	LUBB	TXDOT	0	0	0	0	0	0	0	
Lumcon LA	LUMC	LSU	D2	D2	D3	D3	D3	D3	0	
Lycoming Coun PA	LYCO	LYCO	B51	B51	B51	B51	B51	B51	H48	
*Mac Dill 1 FL	MCD1	USCG	B50	B50	B50	B50	B50	B50	H48	
*Macon 1 GA	MCN1	USCG	B50	B50	B50	B50	B50	B50	H48	
*Macon 2 GA	MCN2	USCG	0	0	0	0	0	0	0	
Managua NU	MANA	NGS	B22	B26	B27	B27	B27	B27	H11	
Marathon FL	MTHN	FDOT	B50	B50	B48	B50	B50	B48	H48	
Marion NC	MARI	NCGS	B50	B50	B50	B50	B50	B50	H48	
McAlester OK	OKMA	OKDOT	B51	B48	B51	B51	B51	B51	B49	
McConnelsville OH	MCON	OHDOT	B50	B50	B51	B49	B48	B48	H44	
McCook NE	RWDN	FSL	Y51	Y51	Y47	Y51	Y51	Y75	72	
McDonald VLB TX	MDO1	JPL	D2	D2	D2	D2	D2	D2	0	
McNeese State LA	MCNE	LSU	D2	D2	D3	D3	D3	D3	0	
Medicine Bow WY	MBWW	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72	
*Medora 1 ND	MDR1	USCG	0	0	0	0	0	0	0	
*Medora 2 ND	MDR2	USCG	0	0	0	0	0	0	0	
*Memphis 2 AR	MEM2	USCG	B50	B50	B50	B50	B50	B50	H14	
Merriman NE	MRRN	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72	
*Miami 3 FL	MIA3	USCG	B50	B50	B50	B50	B50	B50	H48	
Midvale UT	MIDV	SLCSO	B2	B2	0	0	B2	B2	H1	
*Millers Ferry AL	MLF1	USCG	B50	B50	B50	B50	B46	B50	H48	
*Milwaukee 1 WI	MIL1	USCG	B50	B50	B50	B50	B50	B50	H48	
Minaret Summit CA	MINS	USGS	D2	D2	D2	D2	0	0	0	
*Minneapolis WMN	ZMP1	FAA	B50	B50	B50	B50	B50	B50	H48	
Missoula MT	MSOL	MCSO	B48	B51	B51	B51	B51	B51	H48	
*Mobile Point AL	MOB1	USCG	B50	B50	B50	B50	B50	B50	H48	
Monument Peak CA	MONP	SOPAC	D2	D2	D2	D2	D2	1	0	
*Moriches 1 NY	MOR1	USCG	B50	B50	B50	B50	B50	B50	H48	
*Moriches 2 NY	MOR2	USCG	0	0	0	0	0	0	0	
Mt. Pleasant MI	MPLP	MDOT	B24	B26	B25	B25	B25	B24	H24	
Mt Hamilton -CA	MHCB	UCB	D2	D2	D2	D2	D2	1	0	
Muskogee OK	OKMU	OKDOT	B51	B51	B51	B51	B51	B51	B49	
*Myton 1 UT	MYT1	USCG	B50	B50	B50	B50	B50	B50	H48	
Naval Station RI	NPRI	NGS	B25	B26	B25	B24	B25	B26	H23	
Neah Bay - Ch WA	NEAH	UWA	D2	D2	D2	D2	D2	0	0	
Neligh NE	NLGN	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72	
Neodesha 2 KS	NDS1	FSL	B50	B50	B50	B50	B50	B50	H48	
*New Bern 2 NC	NBR2	USCG	B50	B50	B50	B50	B50	B50	H48	
New Jersey In NJ	NJI2	NJIT	B50	B50	B50	B50	B50	B50	H48	
Newberry MI	SUP3	MDOT	B24	B26	B25	B24	B25	B24	H24	
Newport OR	NEWP	OSU	D2	D2	D2	D2	D2	D2	0	
North Liberty IA	NLIB	JPL	D2	D2	D2	D2	D2	D2	0	

Table 3.2 (continued)

Northampton MA	HAMP	NMDPW	B51	B51	B46	B34	B45	B51	H48
Northeast 225TX	NETP	HGCS	D2	D2	D2	D2	D2	D2	0
*Oakland WAAS CA	ZOA1	FAA	B50	B50	B50	B50	B50	B50	H48
*Oakland WAAS CA	ZOA2	FAA	0	0	0	0	0	0	0
Odessa RRP TX	ODS5	TXDOT	0	0	0	0	0	0	0
Ohlone Park CA	OHLN	UCB	D2	D2	D2	D2	D2	D3	0
Okeechobee FL	OKCB	FDOT	B50	B50	B48	B50	B50	B48	H46
Oklahoma CityOK	OKDT	OKDOT	B51	B51	B51	B51	B51	B51	B51
Okolona MS	OKOM	FSL	Y51	Y51	Y51	Y51	Y51	Y69	72
Old Airport TAK	PUO1	BP	B50	B50	B47	B49	B48	B46	H48
*Omaha 1 IA	OMH1	USCG	B50	B50	B50	B50	B50	B50	H48
Oswego Port ANY	OSPA	OSPA	B51	B51	B51	B51	B51	B51	H46
Pacific BeachWA	PABH	CWU	D2	D2	D2	D2	D2	D2	0
Palatka FL	PLTK	FDOT	B50	B50	B48	B49	B50	B48	H48
Palestine TX	PATP	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
Panama FL	PNCY	FDOT	B50	B50	B48	B50	B50	B48	H48
Paul Smith s NY	PSC1	PSC	B26	B26	B26	B26	B26	B26	H20
Penn State PA	PSU1	PADOT	B50	B50	B50	B50	B50	B50	H48
*Penobscot 1 ME	PNB1	USCG	B50	B50	B50	B50	B50	B50	H48
Perry FL	PRRY	FDOT	B50	B50	B48	B50	B50	B47	H48
Pharr RRP TX	PHAR	TXDOT	0	0	0	0	0	0	0
*Pickford 1 MI	PCK1	USCG	B50	B50	B50	B50	B50	B50	H48
Pietown VLBA NM	PIEL	JPL	D2	D2	D2	D2	D2	D2	0
*Pigeon Point CA	PPT1	USCG	B50	B50	B50	B50	B50	B50	H48
Piketon OH	PKTN	OHDOT	B49	B49	B49	B47	B46	B46	H44
*Pine River 1 MN	PNR1	USCG	B50	B50	B50	B50	B50	B50	H48
Pinyon 1 CA	PIN1	SOPAC	D2	D2	D2	D2	D2	1	0
Pittsboro NC	PITT	NCGS	B50	B50	B38	B50	B50	B50	H48
Pittsburgh PA	PIT1	PADOT	B51	B51	B51	B51	B50	B51	B51
Platteville CO	PLTC	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
*Point Blunt 1CA	PBL1	USCG	B50	B50	B50	B50	B50	B50	H48
*Point Loma 3 CA	PLO3	USCG	B50	B44	B50	B50	B50	B50	H48
Point St. GeoCA	PTSG	CWU	D2	D2	D2	0	0	0	0
*Polson 1 MT	PLS1	USCG	B50	B50	B50	B50	B50	B50	H48
*Portsmouth 4 NH	POR4	USCG	0	0	0	0	0	0	0
*Potato Point AK	POT3	USCG	B50	B50	B50	B50	B50	B50	H48
Prudhoe Bay 2AK	PBOC	BP	B49	B49	B47	B47	B48	B48	H46
*Pueblo 1 CO	PUB1	USCG	B50	B50	B50	B50	B50	B50	H48
*Puerto Rico 3PR	PUR3	USCG	B50	B50	B50	B50	B50	B50	H48
Punta Gorda FL	CRLT	FDOT	B50	B50	B48	B50	B48	B50	H48
Purcell OK	PRCO	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
Quincy CA	QUIN	JPL	D2	D2	D2	D2	D2	D2	0
Railroad VallNV	RAIL	UNAVCO	D2	D2	D2	D2	D2	1	0
Raleigh NC	RALR	NCGS	B50	B50	B50	B50	B50	B50	H48
Redman OR	REDM	CWU	D2	D2	D2	D2	D2	D2	0
*Reedy Point 1DE	RED1	USCG	B50	B48	B50	B50	B50	B50	H48
Richmond 1 VA	RIC1	VDOT	B50	B50	B50	B50	B42	0	0
*Robinson PoinWA	RPT1	USCG	B50	B46	B50	B50	B50	B50	H48
*Rock Island 1IA	RIS1	USCG	B50	B50	B50	B50	B50	B50	H48
Saginaw MI	BAYR	MDOT	B26	B26	B26	B25	B26	B26	H24
*Saginaw 1 MI	SAG1	USCG	B50	B50	B50	B50	B50	B50	H48
*Sallisaw 1 OK	SAL1	USCG	B50	B50	B50	B50	B50	B50	H48
Salt Lake CitUT	SLCU	FSL	Y51	Y49	Y51	Y51	Y49	Y75	72
Salt River PrAZ	SRP1	SRP	B23	B24	B24	B15	B19	B24	H19
San Antonio RTX	ANTO	TXDOT	0	0	0	0	0	0	0
San Lorenzo HO	SLOR	NGS	0	0	0	0	0	0	0
*Sandy Hook 1 NJ	SHK1	USCG	B50	B50	B50	B50	B50	B50	H48
Sanford NC	SNFD	NCGS	B50	B50	B50	B50	B50	B50	H46
Santa Elena GT	ELEN	NGS	D2	D2	D2	D2	0	0	0
*Savannah 1 GA	SAV1	USCG	B50	B50	B50	B50	B50	B50	H48
SC00 WA	SC00	UNAVCO	D2	D2	D2	D2	D2	0	0
Scottsdale AZ	COSA	CofS	B51	B45	B41	B49	B51	B45	B51

Table 3.2 (continued)

Seattle	WA	SEAT	UWA	D2	D2	D2	D2	D2	0	0
Seattle Weath	WA	SEAW	FSL	B50	B50	B50	B50	B50	B50	H48
Sedro Woolley	WA	SEDR	UWA	D2	D2	D2	D2	D2	D2	0
Sheldon Wildl	NV	SHLD	CWU	D2	D2	D2	D2	D2	D2	0
SIBY	MI	SIBY	MDOT	B24	B26	B25	B24	B25	B24	H24
Sidney	OH	SIDN	OHDOT	B49	B49	B47	B45	B46	B46	H44
Slater	IA	SLAI	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
Smiths Creek	MI	OKEE	MDOT	B24	B26	B25	B25	B25	B24	H24
Solomons Isla	MD	SOL1	NGS	+3	+3	+3	+3	+3	+3	0
*Spokane 1	WA	SPN1	USCG	B50	B50	B50	B50	B50	B50	H48
St. Croix	VL	VLBVQ	CRO1	D2	D2	D2	D2	D2	D2	0
*St. Louis 3	IL	STL3	USCG	B50	B50	B50	B50	B50	B50	H48
*St. Louis 4	IL	STL4	USCG	B50	B50	B50	B50	B50	B50	H48
St. Paul	MN	STPL	MNDOT	0	0	0	0	0	0	0
*St. Paul 1	WI	STP1	USCG	B40	B50	B50	B50	B50	B50	H48
Stennis	MS	NDBC	FSL	Y51	Y51	Y51	Y51	Y51	Y75	70
Stocker Ohio	OH	STKR	AEC	B50	B50	B50	B50	B34	0	0
*Sturgeon Bay	WI	STB1	USCG	B50	B50	B50	B50	B50	B50	H48
*Summerfield	TX	SUM1	USCG	B50	B50	B50	B50	B50	B50	H48
Surveyor	UT	SUR1	UNAVCO	0	0	0	0	0	0	0
Sutter Buttes	CA	SUTB	UCB	D2	D2	D2	D2	D2	1	0
Syracuse	NY	SYCN	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
Table Mountai	CO	TMGO	NGS	Y13	Y52	Y52	B51	B37	D1	0
Talkeetna	AK	TLKA	FSL	Y51	Y49	Y51	Y51	Y51	Y75	72
Tallahassee	FL	TALH	FDOT	B50	B50	B48	B50	B48	B48	H48
Tecumseh	OK	OKTE	OKDOT	D2	D2	D2	D2	0	D1	D1
Teguciagalpa	HO	TEG1	NGS	D2	D2	D2	D2	D2	D2	0
The Surveyors	AK	TSEA	TSEA	B50	B50	B50	B50	B50	B50	H48
Tiffin	OH	TIFF	OHDOT	B47	B47	B48	B45	B46	B44	H44
Torrance Airp	CA	TORP	USGS	D2	D2	D2	D2	D2	1	0
Trinidad Head	CA	TRND	CWU	D2	D2	D2	D2	D2	D2	0
Tucumcari	NM	TCUN	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
Tulsa	OK	OKTU	OKDOT	D2	D2	D2	D2	0	D3	D1
Tumwater Hill	WA	TWHL	CWU	D2	D2	D2	D2	D2	0	0
Tungsten	NV	TUNG	HSCFA	D2	D2	D2	D2	D2	1	0
U.S. Naval Ob	DC	USNO	JPL	+3	+4	+4	+3	+3	+3	0
University of	IL	UIUC	Univ of Illino	B27	B27	B27	B27	B27	B27	H24
University of	PA	UPTC	PADOT	B50	B50	B50	B50	B50	B50	H48
University of	VA	UVFM	UVA	B11	B18	B15	B20	B14	B14	H13
*Upolu Point	HI	UPO1	USCG	B50	B50	B50	B49	B50	B50	H48
*Upolu Point	HI	UPO2	USCG	0	0	0	0	0	0	0
*Upper Keweenaw	MI	KEW1	USCG	B50	B50	B50	B50	B50	B50	H48
Upsal Hogback	NV	UPSA	UNAVCO	D2	D2	D2	D2	D2	1	0
USACE Field	RNC	DUCK	NGS	D1	D1	D2	B26	B25	B25	H20
USGS Guam Obs	GU	GUAM	JPL	D2	D2	D2	D2	D2	D2	0
*Vandenberg	AF	VAN1	USCG	B50	B50	B50	B50	B50	B50	H48
Vermont Capit	VT	VCAP	VAOT	B51	B51	B51	B51	B51	B51	H48
Vici	OK	VCIO	FSL	Y51	Y51	Y51	Y51	Y50	Y75	72
*Vicksburg 1	MS	VIC1	USCG	B50	B50	B50	B50	B50	B50	H48
*Vicksburg 2	MS	VIC2	USCG	0	0	0	0	0	0	0
Virginia Inst	VA	VIMS	NGS	D2	D2	D2	D2	D2	D2	0
Wasatch, Red	UT	RBUT	Univ of Utah	D2	0	D2	D2	D2	0	0
Washington	NC	WASR	NCGS	B50	B50	B50	B50	B50	B50	H48
Westford	MA	WES2	NGS	D2	D2	D2	D2	D2	D2	0
*Whidbey Islan	WA	WHD1	USCG	B50	B50	B50	B50	B50	B50	H48
White Sands	NM	WSMN	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
*Whitney 1	NE	WHN1	USCG	B50	B50	B50	B50	B50	B50	H48
Wilkes Barre	PA	WIL1	PADOT	B50	B49	B50	B50	B50	B50	H48
Winchester	IL	WNCI	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
Winnfield	LA	WNFL	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
*Wisconsin Poi	WI	WIS1	USCG	B50	B50	B50	B50	B50	B50	H48
Wolcott	IN	WLCI	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72

Table 3.2 (continued)

Wood Lake	MN	WDLM	FSL	Y51	Y51	Y51	Y51	Y51	Y75	72
Wooster	OH	WOOS	OHDOT	B47	B43	B43	B45	B44	B42	H42
Yellowstone	CWY	MAWY	Univ of Utah	D2	D2	D2	D2	D2	1	0
Yellowstone	HID	GTRG	UNAVCO	0	D2	D2	D2	D2	1	0
Yellowstone	HWY	BLWY	Univ of Utah	0	0	0	0	0	0	0
York	PA	YORK	PADOT	B17	B23	B17	B21	B21	D3	0
*Youngstown 1	NY	YOU1	USCG	B50	B50	B50	B50	B50	B50	H48
Yreka Blue	HoCA	YBHB	UCB	D2	D2	D2	D2	D2	D3	0

An asterisk before the site name and state indicates the site broadcasts DGPS correctors.

The letters/symbols refer to the types of files available for that particular day.

H	Observation (hourly)
#	Observation (hourly), Navigation
\$	Observation (hourly), Navigation, Meteorological
N	Navigation
S	Navigation, Meteorological
M	Meteorological
D	Observation (daily)
%	Observation (daily), Navigation
&	Observation (daily), Navigation, Meteorological
+	Observation (daily), Meteorological
B	Observation (daily, hourly)
*	Observation (daily, hourly), Navigation
A	Observation (daily, hourly), Navigation, Meteorological
Y	Observation (daily, hourly), Meteorological

3.3.4 File Naming Convention

The RINEX file naming convention is as follows:

{SSSS}{DDD}{H}.{YY}{T}; where SSSS is the four character site identifier, DDD is the day of year, H is a letter which corresponds to an hour long UTC time block, YY is the year and T is the file type. For daily files, the format would be {SSSS}{DDD}0.{YY}{T}.

Hour long UTC time block identifier (H):

00 01 02 03 04 05 06 07 08 09 10 11 12 13 14 15 16 17 18 19 20 21 22 23	
a b c d e f g h i j k l m n o p q r s t u v w x	
File type	Ending (T)
Meteorological	m
Observation	o
Navigation	n
Summary	s

3.4 CORS Positions and Velocities

3.4.1 CORS Data

The CORS associated files provide 3-dimensional positions and velocities for the L1 phase center of each CORS antenna. These positions and velocities are expressed in each of two different reference frames to meet the needs of the varied groups that use CORS data. One of these reference frames was instituted by the International Earth Rotation Service (IERS); it is called the IERS Terrestrial Reference Frame (ITRF). The other reference frame is the North American Datum of 1983 (NAD 83). For each of these two reference frames, the files present positions in both geocentric Cartesian coordinates (X, Y, Z) and in geodetic coordinates (latitude, longitude, ellipsoid height) for the

GRS80 ellipsoid. Also, for each reference frame, the files present velocities in geocentric Cartesian coordinates (V_x , V_y , V_z) and in local horizon coordinates (northward velocity, eastward velocity, upward velocity) where the vertical dimension is locally oriented normal to the GRS80 ellipsoid. NAD 83 positions given in these files are identical to those contained in the NGS Integrated Data Base.

The IERS publishes revised ITRF positions and velocities annually for a worldwide network of geodetic stations. Each annual solution for these positions and velocities uses observations obtained from various geodetic techniques including GPS, Very Long Baseline Interferometry (VLBI), and satellite laser ranging (SLR). Each new solution not only incorporates an additional year of data, but also the then most current understanding of earth's dynamics.

For the CORS network, NGS expresses ITRF positions and velocities that are consistent with the IERS's solution known as ITRF94 with an epoch date of 1996. The epoch date denotes the date for which estimated positions correspond. An appropriate velocity must be applied to estimate positions for another date. A site's velocity characterizes various forms of crustal motion including the motion associated with plate tectonics, subsurface fluid withdrawal, and crustal loading/unloading. Relative to the ITRF, even points located on

the rigid part of the North American tectonic plate move continuously at rates ranging from 9 to 21 mm/yr in the coterminous 48 states.

Many users of CORS data prefer to position themselves relative to the NAD 83 reference frame that is used widely throughout North America for surveying and mapping activities. Also the U.S. Coast Guard and the Federal Aviation Administration use NAD 83 positions to provide marine and air navigation information. The NAD 83 reference frame is defined so that the North American tectonic plate does not move as a whole relative to it. Thus users can treat NAD 83 positions as unchanging over time except where regional or local crustal motion occurs, e.g., California and southern Alaska, or at locations on a different tectonic plate, e.g., Hawaii.

NGS computes provisional ITRF94 (epoch 1996.0) positions daily for all operational CORS plus for several stations in the worldwide network for which the IERS publishes positions. For these computations, NGS performs a network adjustment with the PAGE4 software (developed by NOAA's Geosciences Lab) applied to 24 hours of GPS observations with a 30-second sampling interval. Each daily solution is constrained by fixing the ITRF94 (epoch 1996.0) positional coordinates for five sites in the IERS worldwide network--Algonquin (Ontario), McDonald (Texas), Goldstone (California), Fairbanks (Alaska),

and Kokee Park (Hawaii). The constrained positions for these five sites are those published by IERS. Each solution is also constrained by fixing the velocity of each site to its NGS predicted value or to its IERS published value.

After a CORS antenna first comes on line, NGS averages the results from at least ten daily solutions to estimate the ITRF94 (epoch 1996.0) position that the agency adopts for official use. This adopted position appears in the associated position file for the station. We can convert these adopted ITRF94 (epoch 1996.0) positions to the adopted NAD 83 (epoch 1996.0) positions via a seven-parameter Helmert transformation. These positional coordinates have an accuracy (95% confidence level) of 2 cm or better in their horizontal components and 4 cm or better in their vertical component.

According to plate tectonic theory, a transformation exists that will convert ITRF94 velocities into NAD 83 velocities. The parameters of such a transformation may be expressed as three rotation rates, one about each axis of a 3-dimensional geocentric cartesian coordinate system. The inverse transformation may be used to convert NAD 83 velocities to ITRF94 velocities. Having estimated appropriate values for these rotation rates, NGS uses the transformation and its inverse to convert velocities between their ITRF94 and NAD 83 values. The

associated position files contain adopted velocities expressed in both reference frames. With a few exceptions these adopted velocities have been predicted because insufficient data exist to estimate precise values for them.

The National Spatial Reference System contains several hundred thousand stations whose NAD 83 positions were determined before the CORS network was established. The accuracy of a differential NAD 83 position between one of these previously existing stations and a CORS antenna depends mostly on the geodetic observations that were used to position the previously existing station. In states where a High Accuracy Reference Network (HARN) has been established, the accuracy of a differential horizontal position between an A-or B-order station and a CORS antenna will almost always be better than 10 cm and usually better than 5 cm. In states where no HARN has been established, the accuracy of a differential horizontal position between a first-order station and a CORS antenna may be only 1 meter, although such accuracies usually will be better than 0.5 meters. These accuracies reflect discrepancies in the positions of the previously existing stations relative to the NAD 83 reference frame. It should be emphasized that positional differences among previously existing stations in a local area will be much more accurate than their common

discrepancy relative to the NAD 83 reference frame. In particular, the accuracy of a positional difference between two existing A-order stations is generally better than one part in 10,000,000 of their interstation distance; between two existing B-order stations, one part in 1,000,000; and between two existing first-order stations, one part in 100,000.

With respect to the vertical coordinate, a HARN station could have an NAD 83 ellipsoid height which is in error by as much as 25 cm, although an error of 10 cm or less is more common. More recent HARN stations have more accurate ellipsoid heights than earlier HARN stations

3.4.2 Transforming Positions

Let the geocentric Cartesian coordinates (X_i, Y_i, Z_i) in meters denote the ITRF94 position for a geodetic site at epoch date E in years. The corresponding NAD 83 position (X_n, Y_n, Z_n) at this same epoch date may be computed by the equations:

$$\begin{aligned} X_n &= T_x + (1+S) \cdot X_i + R_z \cdot Y_i - R_y \cdot Z_i \\ Y_n &= T_y - R_z \cdot X_i + (1+S) \cdot Y_i + R_x \cdot Z_i \\ Z_n &= T_z + R_y \cdot X_i - R_x \cdot Y_i + (1+S) \cdot Z_i \end{aligned}$$

where

$$\begin{array}{lll}
 T_x = 0.9738 m & R_x = (13357 + 42 \cdot (E - 1996)) \cdot 10^{-11} & \text{radian} \\
 T_y = -1.9453 m & R_y = (4872 - 372 \cdot (E - 1996)) \cdot 10^{-11} & \text{radian} \\
 T_z = -0.5486 m & R_z = (5507 + 10 \cdot (E - 1996)) \cdot 10^{-11} & \text{radian}
 \end{array}$$

$S = 0.0$ (unitless).

Note that the three rotations, R_x , R_y , and R_z , are time dependent because the North American plate is moving with respect to ITRF94.

3.4.3 Coordinate Revision

On May 10, 1996, an improved set of positions and velocities were placed in the CORS coordinate files. NGS was able to improve CORS coordinates as a result of three factors:

- the correction of an error in the software used to model positional variation of antenna phase centers
- a change in the ITRF realization used (ITRF94 replaces ITRF93) and the epoch date used for positions (1996.0 replaces 1995.0)
- the use of many more days of data than were previously available

For users who employ NAD 83 coordinates, the most significant changes correspond to changes in ellipsoid height for about half of the CORS antennas. Some ellipsoid heights changed by as much as 17 cm. These large changes are primarily due to the correction of a software error in modeling positional variation of antenna phase centers as a function of satellite location. NAD 83 horizontal positions generally changed less than 2 cm. Horizontal changes primarily correspond to the increase in accuracy realized by using many more days of data than were previously available.

Changes in ITRF coordinates also reflect the effects of replacing ITRF93 with ITRF94 and replacing the 1995 epoch date with 1996. The use of a new ITRF realization and a new epoch date caused only small changes in NAD 83 coordinates because different transformation parameters are used to convert ITRF94 positions to NAD 83 positions than those parameters used to convert ITRF93 positions to NAD 83 positions.

3.4.4 Transforming Velocities

Let the coordinates (V_{xi}, V_{yi}, V_{zi}) in m/yr denote the ITRF94 velocity for a geodetic site whose ITRF94 position is (X_i, Y_i, Z_i) in meters

as expressed in geocentric Cartesian coordinates. The corresponding NAD 83 velocity (V_{xn} , V_{yn} , V_{zn}) may be computed by the equations:

$$V_{x_n} = V_{x_i} + W_z \cdot Y_i - W_y \cdot Z_i$$

$$V_{y_n} = V_{y_i} - W_z \cdot X_i + W_x \cdot Z_i$$

$$V_{z_n} = V_{z_i} + W_y \cdot X_i - W_x \cdot Y_i$$

where

$$W_x = 42 \cdot 10^{-11} \quad \text{radian / year}$$

$$W_y = -372 \cdot 10^{-11} \quad \text{radian / year}$$

$$W_z = 10 \cdot 10^{-11} \quad \text{radian / year}$$

3.5 Differential GPS Using CORS

3.5.1 No-Cost GPS Observations Available Nationally

Significant features of differential GPS are high-accuracy GPS reference stations and GPS positioning data at those stations. In chapter two we showed that we need to:

- use at least two GPS receivers
- set up a GPS receiver at a high accuracy reference station and one or more rover stations

The problems are:

- it is hard to find a high accuracy station around a given area

- it increases project cost

Now we can find a high accuracy reference station by using CORS, the data is available at no cost in most of parts of the country from a system operated by the federal government. In years to come, data of better quality will be available across the country at closer locations and at a higher data collection rate.

The source of the data are CORS, a system that is the outgrowth of real-time differential GPS stations operated in support of marine and air navigation. It acts as a group of high-accuracy GPS receivers, storing observations 24 hours a day and making these observations available, at no cost, for online downloading. Because it will allow after-the-fact GPS data reduction, users can benefit from the system by adding CORS data to data gathered during regular static or kinematic observing sessions. Currently, it is mostly composed of U.S. Coast Guard sites, but other proposed federal installations give the system the potential to grow to several hundred sites.

The goal of CORS is to provide 3D access to the National Spatial Reference System, accurate to within 2- 3 millimeters. The new geoid models developed, GEOID96 and GEOID99, will play a large part in obtaining required vertical accuracy. But it will be several years before

observations are made on a local benchmark to achieve high vertical accuracy.

3.5.2 Expected to Achieve 1 Centimeter Accuracy

Currently, all of the CORS sites are accurate to within 3 centimeters, horizontally, and 5 centimeters, vertically, relative to each other but NGS expects to have this down to 1 centimeter, horizontally, and 2 centimeters vertically. With better and more effective ionosphere models, even single frequency receivers could achieve better accuracy.

Actually, CORS not only offer the differential positioning capability needed for surveyors, CORS data is also being used to control aircraft performing aerial photogrammetry. Another use has been found that might aid in weather forecasting. Because the positions of the CORS sites are known, meteorologists are able to use the data to estimate water content of the troposphere and use that information in the development of weather and climate models.

The CORS Mission includes the assurance that all navigation and positioning in the United States is performed using a common coordinate system, providing users with data for after-the-fact positioning, permitting multi-use of CORS sites established by different

groups, and promotion of CORS standards to facilitate multi- use. Recently, some of the data in CORS system was obtained at a five-second sampling rate and placed in hourly files made available within 10 minutes after the hour. Other data was gathered at 30-second rates, available within a few hours of the beginning of the next day. Users could interpolate 5-second data from 30-second data. Currently, data for the past three weeks is available online. Beyond that, data is archived on CD-ROMS that are available through normal distribution channels.

NGS has established two different ways to gather the data. The first involves the use of a personal computer at the station. Files are automatically downloaded to the central data facility at NGS in Silver Spring on an hourly basis. Several days of observations are stored on the computer at the station. The advantage of this system is that the data can be retrieved from the remote computer in case there is a problem at the central data facility. The disadvantage is that the remote computers can, and do, malfunction, and personnel have to be available to deal with the inevitable malfunctions. The second method of data retrieval, the one employed to gather data from the Coast Guards sites, involves dedicated phone lines over which packets of data are sent to the central data facility. Data is sent real-time, and there

are no computers or storage devices at the site. The disadvantage of this system is that if there is a problem at the central facility, all the data are lost. This data transmission is not to be confused with the primary mission of the Coast Guard stations, that being the radio transmission of real time differential corrections for navigation purposes. NGS is currently implementing a redundant system of receiving computers at the central facility to ensure that data received from the Coast Guard sites will not be lost if a computer goes down.

3.5.3 Central Data Facility

The central data facility performs five primary functions:

- conversion of the data to a common format (RINEX)
- quality control of the data
- determination and monitoring of station positions on a daily basis
- creation of files for distribution
- data archiving

With a five second sampling rate, each station will produce about 5 megabytes of data per day in a compressed format. One hundred stations would therefore require handling half a gigabyte per day

which, although not an unreasonable amount, requires the process to be almost completely automated. This amount of data is equal to almost one CD-ROM per day. Some users, notably the hand-held and GPS-in-the-plane photogrammetry segments and airborne surveying, have requested a one-second to 0.1 second sampling rate, but this would result in each station creating more than 25 megabytes of data per day.

3.5.4 Precise Orbital Information

Precise orbital information could be obtained from the Coast Guard, NGS or IGS. This information is available within one week after-the-fact and is importable by most GPS processing software. Improved techniques have allowed the location of the satellites to within 10 centimeters of their true orbital position and this precise orbital information will generally improve the accuracy of baselines over 100 kilometers in length. Baselines several thousand kilometers in length are being measured to within one centimeter horizontally. Once the CORS sites are accurate to within a centimeter, if the data is unavailable from one CORS site, the user can wait 4-5 days even 15 days for the precise orbits and then use any other CORS site to achieve equal accuracy.

3.5.5 CORS DGPS and HARN GPS Data Analysis

Procedure

Differential GPS (DGPS), which is a GPS technology for high accuracy observations, can take measurements down to a centimeter. Besides its high, three-dimensional accuracy, GPS can be used in all weather, day and night, and all over the world, at very low cost. GPS has brought about a revolution in surveying technology

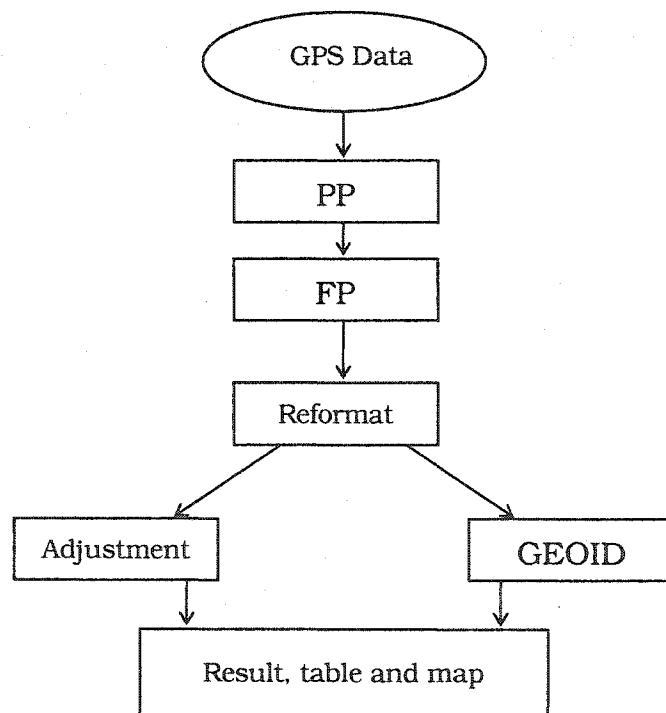


Fig. 3.4 HARN GPS Data Analysis Procedure

To obtain a high accurate DGPS solution. The right and accurate data processing plays a very important part. Fig. 3.4 shows Iowa HARN GPS data processing and analysis procedures. It consists of following procedures:

- GPS data: HARN GPS network planning, data collection.
- PP – Per-Processing: GPS data pre-processing, which includes data reformatting, checking, joining, and merging.
- FP – Final Processing: fixing cycle, ambiguity search and final solution.
- Reformat: datum transformation, vector computation, and reformatting.
- Adjustment: network adjustment, accuracy analysis, coordinate system transformation.
- Geoid: Geoid model evaluating, analyzing, and re-modeling
- Results, Table and Map – obtained final results for Iowa HARN project.

CHAPTER 4. HARN GPS DATA PROCESSING

4.1 Introduction

The HARN data processing consists of two parts. One is the data processing, and second is GPS network adjustment. The processing in HARN project we use **OMNI**, which is an official NGS program to process all static GPS data. **OMNI** is a suite of over 50 programs designed to process data collected by carrier-phase receivers from the satellites of the Global Positioning System (GPS). **OMNI** was originally developed by the Geodetic Research and Development Laboratory of the National Geodetic Survey (NGS) (now with the Office of Ocean and Earth Sciences). In June 1989 the Space and Physical Geodesy Branch at NGS adopted OMNI for the office and field processing of all static GPS data collected by NGS.

OMNI accepts RINEX files as input. It also allows users to convert **RINGO** format into RINEX 2 format. **OMNI** could process data in static, kinematic, and rapid static modes.

GPS processing with **OMNI** consists of the sequential steps [fig.4.1] shown below.

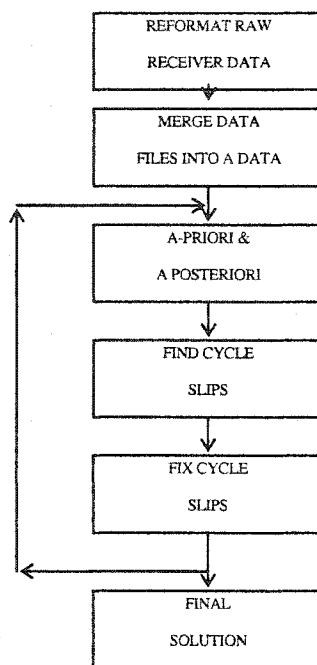


Fig. 4.1 OMNI Data Processing Steps

The first step reformats the data recorded by the receiver into a standard format used by **OMNI**.

The second step merges all the separate reformatted data for

each station, along with satellite position and velocity information, into a three-file database. In addition to the observations, this database also contains computed quantities needed at each epoch, thus saving repetitive calculations and providing a convenient format to contain all the information for a single observing session.

The next three steps, also known as cycle slip fixing, involve the iterative use of four programs. **GPS22** is run on the first database to generate a-priori double difference phase residuals. These residuals are recorded in the DDR (Double Difference Residuals) plot file. Program **CHKDDR** uses this DDR file to find cycle slips and to generate a list of corrections. The **EDATA** program takes this list of corrections and applies them to the database to create a new database. **GPS22** is then run again. This cycle is repeated until the **CHKDDR** program finds no further cycle slips. An a-posterior **GPS22** solution is then run as a final check. The a-posterior residuals are stored in the PFR (Post-Fit Residuals) plot file. Program **CHKPFR** uses this plot file to check for any remaining cycle slips. The entire fixing process is done first for L1 and then for L2.

In the last step, after all the L1 and L2 cycle slips have been repaired, the user can run final L1-fixed or ionospherically-corrected solution. For the final solution the NGS G-FILE information is output

to allow for a later network adjustment of all independent vectors included in the project. To create the final project GFILE the program **MAKEGF** is run outside of OMNI [33].

4.2 Re-Format Raw Data

The project like HARN, we have to use many receivers at the same time to collect data. Data collected by these GPS receivers is recorded in a wide variety of formats and with variations in the individual definitions of time, range and phase, newer receivers may record lock time, and signal noise ratio (SNR). The subsequent processing steps of **OMNI** require that all the incoming data be expressed in a standard format with specific definitions of the fundamental GPS observables; time, phase and range. We can convert these different formats to a standard format RINEX. In this research we use the tool in OMNI to convert the data.

4.3 Merge a Database

The **OMNI** programs are currently designed to include up to 12 stations and 24 satellites in an observing session (20 stations are allowed by using an 8-megabyte version of **GPS22**). To avoid addressing the station data files individually and make the program

more easily to process the data, all the station data are merged into a database. This provides a convenient way to refer to all the stations in a session as a single entity. It also provides a place to include parameters required for solutions so that they need not be repetitively calculated.

9218.06		CURRENT MERGE SETUP								v3.21		
DB NAME:	START:	DOY	HR	MM	SEC	STOP:	DOY	HR	MM	SEC		
INTERVAL:	ELV LIM:	SU IDs: 2 3 6 9 11 12 13 14										
BC ORB FILE:	ORBIT TYPE:				EPHEM FILE:							
MASTER CLOCK: Use Start & Stop Times												
STATION SUMMARY												
#	NAME	EDIT	SOLVE	SUCLK	RCUR	#	NAME	EDIT	SOLVE	SUCLK	RCUR	
1	ATHY305A	YES	YES	BOTH	TRIM							
2	ASTW305A	YES	YES	BOTH	TRIM							
3	OPTN305A	YES	YES	BOTH	TRIM							
4	GORP305A	YES	YES	BOTH	TRIM							
5	MDPT305A	YES	YES	BOTH	TRIM							
ANALYST'S NAME:											RUN	QUIT

Fig. 4.2 Data Merge Setup

Several quantities must be defined in order to create this database

- The database must have a start and a stop time and station data falling outside these limits will not be included.
- An elevation cutoff will be used to set an elevation limit below which data will not be included. Satellites or stations which are subsequently found to be bad may also be excluded from the database.

- The database contains observations and calculations at each epoch of the database.

Database will consist of three files:

- xxxxHD.DAT: ASCII header file containing global information for the stations and satellites.
- xxxxOR.DAT: BINARY file containing the satellite positions and velocities for each epoch.
- xxxxDT.DAT: BINARY file containing the phase and range observations along with edit and tropospheric information.

STATION PARAMETERS include station position (X, Y, Z), antenna offsets (N, E, UP, L1-L2), meteorological measurements (T, P, H), and receiver type. These values are obtained from operator entries in the field or in the case of the XYZ position, from receiver generated solutions. These values are notoriously incorrect and should be examined very carefully for each station.

The **AUTO EDIT** field specifies whether the automatic cycle slip editing feature of **MERGE** needs to be turned on. This would normally be done for static receivers but should never be done for mobile receivers because the phase changes due to receiver motion would be interpreted as cycle slips and fixes would be attempted. This cycle slip

fixer uses a two-point extrapolation technique. Depending on the clock stability, the accuracy of the station positions and the interval between observations, these corrections may be good to a few or a few hundred cycles. For the **AUTO EDIT** field the user must choose either **YES** or **NO**.

The next step in the processing, cycle slip fixing, works best with a good estimate of the station positions. If the user believes his a-priori positions are worse than a meter or so, **MERGE** offers an option for a triple difference solution (L1 only) for station positions as the data base is being created. This option is exercised by selecting YES on the **SOLUTION STATUS** line. In this case the triple difference solution XYZ values will be written to the data base header file. If not exercised, by selecting **OMIT**, the XYZ values given above and written to the *.POM file will also be written to the *HD.DAT header file. If good coordinates are available from previous solutions for this station, these coordinates are entered above for inclusion in the *.POM file. The triple difference solution is not exercised. Using identical coordinates for a station observed many times during a campaign will simplify position comparisons later on.

NOTE: When using receivers that collect at fractional parts of a second, or when mixing receivers that collect at different intervals, it is

good practice to use ENTER START TIME to set the start time to an even minute. This minimizes the amount of time tag shifting that must be done and insures that data taken at different intervals will not be skipped over accidentally

Epoch times in the database are derived from the start time plus some integer multiple of the interval. The station observations closest to the data base times are selected for inclusion in the database. If the observation times are not exactly coincident with the data base time, the phase and range values will be corrected by the time difference multiplied by the Doppler rate and a continuously estimated clock drift for each receiver. The accuracy of this correction depends on the accuracy of the a priori station coordinates, the satellite orbits and the amount of the shift. Generally, shifts of a second or less can be done with negligible degradation of accuracy. This is the feature that allows different receiver data with slightly different time tags to be combined.

The ORBIT TYPE either BROADCAST or PRECISE is chosen next. Precise ephemeris files must have the extension *.EPH and must be in the NGS standard product 1 format (SP1).

A precise ephemeris can be used at any time in the data processing to replace the satellite coordinates currently in the database by using the utility program **NUORB**. Thus, a session for which a

precise ephemeris is not yet available can be started quickly using the broadcast orbits. The database can be made and edited and preliminary solutions obtained to evaluate data completeness and quality. When the precise ephemeris becomes available, **NUORB** can be used to create a new database from the current one but with a *OR.DAT database file that is now derived from the precise ephemeris. The final static solutions can then simply be re-run. In this way also, identically edited data may be compared between different sources of satellite ephemerides.

While **MERGE** is running certain messages will be displayed and updated as the data is read and put into the database. The meaning of most of these messages is obvious. One message that may need attention is the "BAD START ..." message. This message is related to the **AUTO EDIT** selection made earlier while specifying the station options.

- it assumes a cycle slip exists and a correction is estimated and entered into the database.
- the clock stability is worse than the tolerance set by **MERGE**;
- the a priori station coordinates are grossly incorrect.

MERGE filters the data using certain cycle slip and slope threshold. Grossly inaccurate starting values for station positions may result in very little or no data getting through this filter. Generally, if the a-priori coordinates are within a few kilometers of the correct position (for the non-reference stations), the triple difference solutions should work.

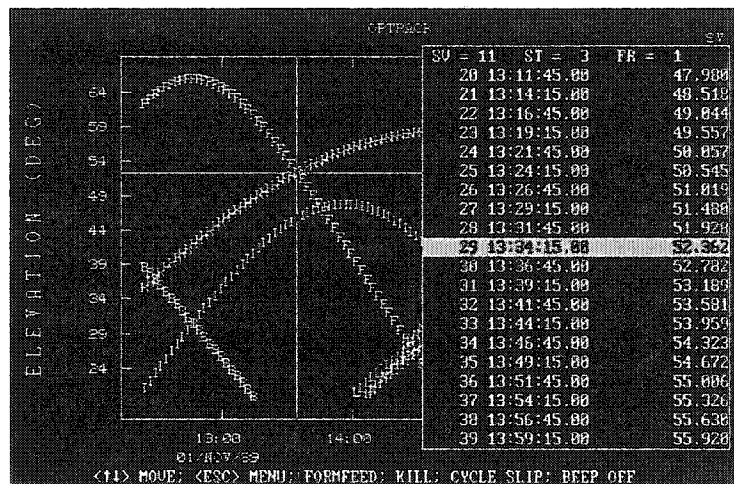


Fig.4.3 Plot the Satellite Elevation

At the end of **MERGE** the user may **QUIT** and return to the main **OMNI** menu or **PLOT** the **MERGE** plot files.

The satellite elevations for the stations are shown above [Fig.4.3]. The ELV plot files and the PXC plot files should be used together to choose the reference satellites. It is best to try and choose the 2 or 3 highest satellites as reference satellites. The **List** option in the **PLOT**

program can be used to find the cross-over points of the highest satellites. The times of these cross-over points are used as the start times of the different reference satellites. The reference satellites, the day-of-year, and the start times (H:M:S) for this session are entered. These values will be used later when running program **GPS22**.

4.4 Cycle Slip Fixing

After **MERGE** has been run, we can start cleaning cycle slips. At **OMNI** this is done using two programs called **CHKDDR** and **CHKPFR**.

CHKDDR (which stands for **CHeck** the **DDR***.PLT** plot file) is a program which uses the *a-priori* double difference residuals output by the **GPS22** program to generate a list of possible cycle slips (i.e., the epoch where they occur and the estimated size of the slip). This list of cycle slip corrections is written to a file which has an extension *.EDT.

CHKPFR (which stands for **CHeck** the **PFR***.PLT** plot file) is a program which uses the *a-posteriori* double difference residuals output by the **GPS22** program to generate a *.EDT file.

There are many ways to approach the problem of fixing cycle slips in carrier phase data. Most manufacturers of GPS equipment strive to make their cycle slip fixing software completely automatic to increase productivity. This is usually possible for one type of receiver,

especially if that receiver happens to be a dual frequency P-code receiver. For C/A-code receivers what usually happens is that the automatic fixer will work well for good data sets, but will delete large amounts of data or perhaps even fail for bad data sets. This is because extremely noisy data can have L2 double-difference residuals whose drifts will approach the half-cycle level. Thus, making a cycle slip fixer that works well for all different types of receivers tends to be difficult.

A certain emphasis on "flexibility" then has been required in the NGS cycle slip fixing process. **CHKDDR** and **CHKPFR** are two programs that have grown out of this type of environment. They are not completely automatic cycle slip fixers, but instead require some final interactive decisions to be made by the user.

4.4.1 Advantages of the Interactive Approach

The analyst knows exactly what his data is like during every step of the processing, and sometimes even horrible data sets can be selectively edited to get a good solution.

CHKDDR and **CHKPFR** can be used on single frequency as well as dual frequency receivers. The **disadvantage** is that for bad data sets salvaging as much data as possible will usually take more time.

The edit files (*.EDT files) created by both of these programs must be examined by the user if the jumps being searched for are small (under 0.8 cycles) so that cycle slips are not repaired incorrectly across large gaps or across periods of extremely noisy data. Once verified by looking at the plots, the cycle slip corrections can be applied to the database using the **EDATA** program. **EDATA** takes the cycle slip corrections in the *.EDT file and adds the given number of cycles in each correction to the L1 and L2 edit columns in the *DT.DAT file of the database.

CHKDDR and **CHKPFR** were originally developed as utility programs for use with the dual frequency, C/A-code receivers operated by the NGS field crews. They work on the assumption that for all C/A-code, dual frequency data the L1 data will be cleaned up first and then the L2 data. The cycle slips in the L1 data are cleaned up first because for C/A-code receivers the L1 wavelength is longer (19 cm) than the squared L2 wavelength (12 cm). Obviously, for dual frequency, P-code receivers, where the wavelength is a full 24 cm, the L2 cycle slips are much easier to fix. However, even for P-code receivers the analyst should still clean the L1 data first because **CHKDDR** and **CHKPFR** will always blame the jumps in the ionospheric-free residuals on the L2

data. Both programs will allow only "full" cycle slips in L1 and L2 for P-code receivers

Choosing the **STATIC SOLUTIONS** option from the **OMNI** will bring up the **GPS22** setup page. The first time this option is selected the entries seen on the next screen will be blank or set to default values. After **GPS22** has been run once, the previous setup instructions will be saved and displayed so that only a few changes will need to be made for subsequent runs.

The user must now select the type of **GPS22** solution either **SOLUTION** or **A PRIORI SOLUTION** will generate a complete a-posteriori solution, solving for all unknown parameters.

A PRIORI will generate plot files of the a priori double difference residuals only.

4.4.2 The Steps of the Cycle Slip Fixing Process

Choose **A PRIORI**.

Set **FREQUENCY**: L1, L2, **IONOSPHERIC-FREE**, or **WIDELANE**.

The **START** and **STOP** times for the processing are specified by the **DOY** (day of year) and the hour, minute and second. They default to

SELECT A SINGLE REFERENCE SATELLITE -- for short (1-2 hour) sessions where only one reference satellite will be used. When that satellite is present at the reference station, it will be used as the reference satellite and when it is not present, the satellite list is searched for the first good satellite at the reference station, beginning with the lowest PRN number. The satellite located becomes the reference satellite for that epoch. By differencing the integer ambiguities within the program, the user specified reference satellite remains the reference even when it is no longer present in the observations.

SELECT MULTIPLE REFERENCE SATELLITE -- for longer sessions because it allows the user to choose the highest satellites to use as reference satellites. Unlike option one, when these specific satellites are not available during their specified time slots, the program will skip over these epochs. This is usually not a problem since the highest satellites are usually those that maintain lock the best.

The GPS22 a priori solution will make one pass over the data to compute the a priori residuals and then stop. After it is done, the CHKDDR program can be run. The user is asked to enter the criterion to be used for finding cycle slips. Since this is the first time any L1 cycle slips are being detected, it is possible that the data may have

many large cycle slips (>1000). These large slips can make even the good data look bad. Thus, it will be better to correct for all of the large cycle slips first. Experience has shown that correcting all cycle slips larger than 10 cycles for the first iteration will make easier to find the remaining slips. For the cycle slip detection criterion the number "10.0" is typed in. The program next asks for the maximum sized gap across which it will still be legal to fix cycle slips. If there are a large number of residuals missing (a gap) then the analyst will have to decide what the trend of the residuals was before the gap and whether the data after the gap agrees with this trend or has incurred a cycle slip at some time during the gap. Gaps smaller than 10-15 minutes are usually not a problem. At a 15-second interval, 20 epochs = 5 minutes. As a starting out value for the sample data set the number "20" is typed in. (if any larger gaps are discovered this number can be increased and CHKDDR run again). It is important for the analyst to know if any large gaps (>30 min.) are present in the data because these can sometimes make a data set completely unusable (i.e., the gap can be too large to reliably fix a cycle slip across when the data is noisy).

After entering the number of epochs allowed for fixing across a gap, the CHKDDR program begins running. It starts scanning through the *.PLT file, compares neighboring L1 double difference residuals and

displays on the screen any jumps in the residuals that are over 10.0 cycles. The information sent to the screen includes the time of the cycle slip, the satellite PRN number, the station number, the last epoch number before the slip, the epoch number where the slip was noticed, the actual double difference residuals before and after the slip, the difference between these residuals, and finally the cycle slip correction that will be needed to get the before and after residuals to agree. This cycle slip correction, shown in the far right column, will be rounded to the nearest full cycle (nearest half-cycle for C/A-code L2) and written to an edit file called *.EDT.

The jumps in the residuals (cycle slips) that are shown on the screen are sometimes NOT written to the *.EDT edit file for three reasons.

(1) If a gap is larger than the maximum gap criterion the cycle slip is flagged with two asterisks and not written to the *.edt file.

(2) If a slip is sensed at a point where the GPS22 program switched reference satellites, then more than likely that jump in the residuals is caused by orbit error and not by a true cycle slip. These satellite switch times are marked with a single asterisk and are not included with those cycle slip corrections written to the *.EDT file.

(3) The last reason a jump might not be saved to the *.EDT file is because it is not big enough. CHKDDR has its own conservative thresholds for selection; a L1 residual jump must be larger than 0.8 cycles before it is rounded up to 1.0, and a C/A-code L2 residual jump must be larger than 0.4 cycles before it is rounded up to 0.5 to be a half-cycle slip. Nothing below 0.4 cycles is ever rounded off and automatically written to the *.EDT file.

After the B-database is created using EDATA, it is time to run another a-priori GPS22 solution to repair the cycle slips smaller than 10.0 cycles. The GPS22 menu will now search the directory for the *HD.DAT files denoting any existing complete databases.

One iteration of fixing cycle slips which includes creating a residual file using GPS22 and the A-database, examining this plot file from GPS22 using CHKDDR [Fig. 4.6], correcting the cycle slips to create a new database using EDATA, and finally running the new B-database through GPS22 to create a new DDR plot file has been described. These are the steps used to first clean up all the cycle slips in L1, and then all the cycle slips in L2.

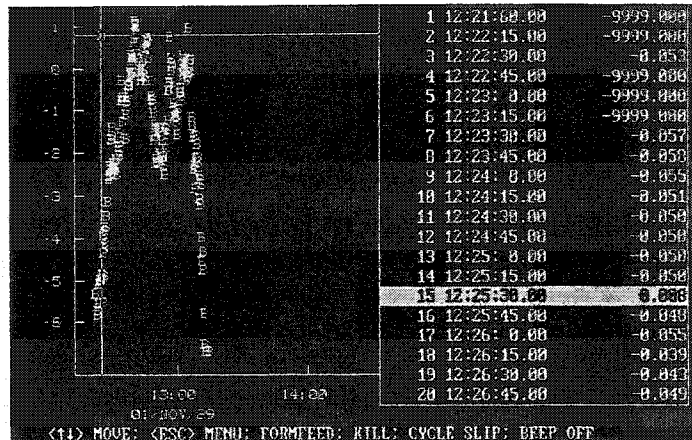


Fig. 4.5 Correcting the Cycle Slips

To clean up the final few slips and outliers by hand we will run an A-POSTERIORI **GPS22** solution on the latest database is used. In the **GPS22** setup menu change the PROCESSING MODE: from A-PRIORI to SOLUTION.

The PLOT program can be used to generate cycle slip corrections and delete commands. Outliers are not corrected (since they are smaller than 1.0 cycle) but are instead deleted. Deleting epochs can be done using the **KILL** option in the **List** feature in program **PLOT**. With the crosshair on epoch 15 the letter "K" is pressed. A frequency submenu will appear if this is the first time PLOT is being run. The L1 is selected and entered. The word "KILLING" will appear in the lower right corner of the screen as shown above. The highlight bar is moved down to the last epoch to be deleted (which in this case is still epoch 15

so the highlight bar remains on the same epoch) and "K" is pressed again. The word KILLING will disappear signifying that one delete command has been written to a file called PFR*.LOG. Because the data appears to go up and down erratically before this gap, there is a good chance that this 0.67 cycle jump could be considered a +1.0 cycle slip (for PFR plots a positive correction used to bring down the data to the right of a slip). To create an edit command for this cycle slip epoch 155 is first highlighted and "C" typed for cycle slip. If the "PLOT" program is first started then L1 frequency is chosen.

The word FIXING will appear in the lower right corner of the plot screen signifying that the user is testing two epochs for a cycle slip. Next, epoch 172 is highlighted and "C" typed again.

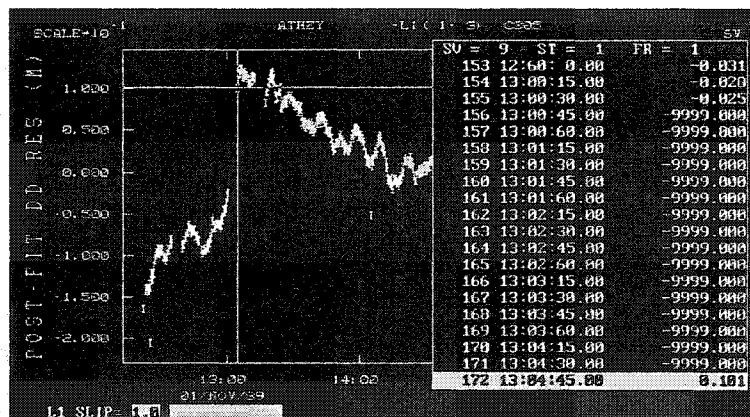


Fig. 4.6 Fixing Cycle Slips

The screen will show L1 SLIP= 0 because the gap is smaller than 1.0 (0.67). Because we want to force this slip correction to be +1.0 we type 1.0. The screen will then appear as shown above [Fig. 4.6].

After going through all the plots for all the satellites at each of the four non-reference stations there should be one PFR*.LOG file containing the edit commands. From this a-posteriori solution the *.PLT file is plotted and all of the cycle slips in L1 are now cleaned. The plots will still show a great deal of variation and scatter due to the effects of the ionosphere. Once we clean up L2 and run an ionospheric-free solution, these plots will look considerably better.

Now that all the L1 cycle slips have been fixed, cleaning up the L2 cycle slips begins. We do this by running **GPS22** using either the L2 frequency or the ionospheric-free combination of the L1 and L2 frequencies. For short to medium baselines (» 0 to 40 km) the L2 frequency will work fine (unless the ionosphere was active during the session). If the ionosphere was active, or if the baselines are longer than 40 km, then it is best to use the ionospheric-free combination to clean up the L2 cycle slips. **CHKDDR** and **CHKPFR** have been designed specifically for using the double difference residuals from an ionospheric-free solution. When a discontinuity or jump is seen in the ionospheric-free residuals, it can be caused by a cycle slip in L1, a

cycle slip in L2, or a combination of cycle slips in both L1 and L2. However, if L1 is carefully cleaned up first, as we have done for the data set, then any jumps seen in the ionospheric-free residuals must be caused by slips in L2.

4.5. GPS22

4.5.1 Final Solution of DGPS

OMNI will permit a variety of options for obtaining GPS solutions. These include flexibility in the selection of frequencies, treatment of the station clock errors, and solving for tropospheric scale height corrections. **OMNI** also allows for the adjustment of the Keplerian elements for each satellite observed during the session. This feature provides the greatest possible accuracy for baselines over 100 kilometers when fiducial station data are also included in the database.

When the **STATIC SOLUTIONS** field is highlighted in the **OMNI** main menu, the **GPS22** menu program (GPSSET.EXE) is called. Once the user enters the information requested by this menu, he is ready to run the static solution program called **GPS22**. This program is the real "heart" of the **OMNI** software package. It is this program that

computes the actual baseline vectors and station coordinates that are the final goal of any GPS user. Choosing **STATIC SOLUTIONS** runs the **GPS22** program, the two names will be used interchangeably in this program. We shall now describe how to use the **GPS22** menu to enter the **GFILE** information for an ion-free, fixed-integer solution on the last database made for the data set.

NOTE: Before generating final ion-free, float-integer solution using the H-database the **INTGR** file must be deleted. The way the **GPS22** program fixes integers is by reading the **INTGR** file. Whenever it reads an L1 or L2 integer (from the first and second columns of the **INTGR** file) and sees a fractional part that is .00 or .50 exactly, that integer is held fixed. Occasionally integers solved for in the L1 (or L2) a-posteriori solution may end in .00 or .50 by coincidence. To prevent these from being designated as fixed in the **SAVIT** file, the **INTGR** file is totally deleted. Then the ion-free solution will be forced to be an all-float solution.

4.5.2 Processing Mode

This **SOLUTION** will generate a complete a-posteriori solution, solving for all unknown parameters.

A PRIORI will generate plot files of the a priori double difference residuals only. For generating the final ion-free solution, with GFILE

GPSSET v3.22		STATION LIST	
DATABASE NAME: ???? 9218.23		#	NAME STAT CLN HGT
PROCESSING MODE:	GFILE	1	
FREQUENCY:	OPTIONS	2	
DOY HR MN SEC			
DATABASE NAME			
H305HD.DAT	INSTRUCTIONS: The following is a list of database header file(s) found in your current directory. Select a single header file to be used for your processing. The first four characters of the file you select will be used as the database name. Press the ENTER key to make a selection. Use the UP/DOWN ARROW keys to move the highlighter among the various file names.		
RUN	QUIT	ADJUSTED SV ARC ELEMENTS:	
ANALYST'S NAME:			

Fig. 4.7 Generating the Final IONO-free Solution

information, see [Fig.4.7] in computer screen. To get a better and final solution, you have to set some other options too. The other options related to the processing mode are:

Frequency:

- L1 will compute a solution using only the L1 carrier phase data.
- L2 will compute a solution using only the L2 carrier phase data.
- ION-FREE will combine the L1 and L2 data phase data to correct for the effects of the ionosphere.

- WIDE-LANE will combine L1 and L2 (L1-L2) to create an 86-cm wavelength observable (for receivers with full cycles in L1 and L2).

Satellite ephemeris used in the solution

- the broadcast ephemeris -- BROAD
- precise ephemeris -- NGS.

Accuracy of the Ephemeris

- for the broadcast ephemeris (1.0 meters)
- for a precise ephemeris (0.2 meters).

Coordinate System used in the satellite ephemeris:

- WGS84-B use this code for the Broadcast Ephemeris,
- WGS84-P use this code for the NSW or DMA Precise Ephemerides,
- ITRF89 IERS Terrestrial Reference Frame 1989 (epoch 1988.0),
- PNEOS90 Provisional NEOS 1990 (March 1991),
- NEOS90 N National Earth Orientation Service 1990 (July 1991),

- ITRF91 IERS Terrestrial Reference Frame 1991 (after GPS week 658).

Station Status submenu

- SLV (solve for this station's position)
- REF (use this station as *the* reference station)
- FID (use this station as a fiducial station when satellite arcs are adjusted)
- OMIT (omit this station from the solution)

Tropospheric scale height is to be adjusted at this station by selecting

- SLV (solve for the scale height)
- FIX (do not solve for the tropospheric scale height)

Stations for which only approximate tropospheric data have been included in the database should have their scale heights adjusted. However, both ends of a baseline should not be simultaneously adjusted unless the baseline is long enough for the troposphere to be uncorrected. This distance is usually determined empirically (» 100 km). For small and moderate size networks like the test data set, the

HGT is fixed at the reference station and SLVed elsewhere providing relative scale heights of each station with respect to the reference station.

The information of SAVIT file:

- the date and version number of the GPS22 program which produced this SAVIT file
- the date and time the solution was run
- the name of the person who ran the solution.
- the GPS22 SETUP SUMMARY which echoes all of the information gathered on the GPS22 setup page
- the DATABASE HISTORY
- when MERGE was run to create the A-database
- the start and stop times of the database
- type of ephemeris was used to create the database
- the SETUP SUMMARY which lists the different types of unknown parameters being solved for
- REFERENCE SATELLITE SCENARIO shows the reference satellites used in the solution.
- the lower triangular part of the correlation matrix, the matrix starts at the top with the integers

- the tropospheric scale height parameters
- lists the XYZ coordinates
- After the correlation matrix comes the
- OVERALL RMS OF FIT for the solution (in meters)
- the actual solution results
- station's name
- XYZ coordinates for the antenna L1 phase center
- the antenna offsets from the monument: North, East, Up
(vertical antenna height)
- the separation between the L1 and L2 phase centers (L1-L2)

Cartesian coordinates (XYZ)

- geodetic coordinates (Latitude, Longitude, and Ellipsoid height in meters)
- the standard deviations of these coordinates
- integer ambiguity parameters and their sigmas

For the HARN project of IOWA, a 0.015 meter or lesser RMS obtained from a solution is a satisfactory result. For shorter baselines, under 20 km, RMS can be of 6 to 9 millimeters. For baselines longer than 100 kilometers, the overall rms can be between 0.02 and 0.03 meters. These "ballpark" estimates will vary depending on how noisy

or clean the data is and how much the ionosphere and weather conditions affected the solution. The individual rms values for each satellite at each station are also given. By comparing these rows and columns the user can see if any particular station or satellite had problems.

It should be noted that for an iono-free solution, the ambiguity parameters would not be close to integers because L1 and L2 have been multiplied by non-integer constants and then subtracted (iono-free = $2.546*L1 - 1.984*L2$). The tropospheric scale height is given next in terms of a percentage. A +0.01 scale height parameter means that an extra 1% was added to the tropospheric model correction to account for unknown atmospheric effects. A -0.03 means an extra 3% was subtracted from the values given by the tropospheric model. Finally, the "n-1" baselines are given with respect to the current station. The baseline lengths are also listed. These baselines are derived from XYZ coordinates computed for each of the unknown stations, and the XYZ coordinates held fixed for the reference station.

4.6 Fixing Integers

Once a cycle slip free, iono-free float solution has been obtained, the steps described below are followed to fix the L1 and L2 integer ambiguities. Two programs to be used

GPS22 -- the program that is run under the **STATIC SOLUTION** section of the **OMNI** main menu

BIAS --a program found under the **UTILITY PROGRAMS** section of the **OMNI** main menu. A text editor is also required to modify the contents of the INTGR file and the *HD.DAT database file.

The STEPS followed are:

1). Each line in the INTGR file created by **GPS22** has six columns: station name, PRN number, L1 integer ambiguity (cycles), L2 integer ambiguity (cycles), iono-free ambiguity (meters), and the wide-lane integer ambiguity (meters). The process is begun with an INTGR file that has only the iono-free column (the 5th column) filled in with actual ambiguities. If the L1 and L2 columns (3rd and 4th columns) are filled in from previous L1 and L2 solutions, then the INTGR file is erased and a new one created by running **GPS22** for an iono-free float solution.

2). Using the adjusted XYZ station coordinates found in the SAVIT file from the iono-free float solution, the XYZ coordinates for all

of the unknown stations in the *HD.DAT database file are entered using the text editor. This will make fixing the integers easier, especially when baselines are long (100 KM).

3). In the menu for **GPS22** all of the station XYZ unknowns are turned off (i.e., for all unknown stations change "SLV" to "FID" in the STATION LIST submenu under the heading STAT(status)). This will cause the coordinates for the unknown stations to be fixed to the values which are entered into the *HD.DAT file as specified in STEP 2.

4). Also, in the same STATION LIST submenu is a heading called HGT (tropo scale height). For medium baselines (20 to 100 km) it is normal to solve (SLV) for the tropospheric scale height for each of the unknown stations and to fix (FIX) the tropospheric scale height at the reference station -- this provides relative scale heights for each station with respect to the reference station. For longer baselines (over 100 km), where the troposphere becomes uncorrelated at both ends of the baselines, the reference station should solve for its own tropospheric scale bias; in this case FIX to SLV must be changed for the reference station. For very short baselines (under 5 km) you can set all the tropospheric scale heights not to FIX, because the double differences used in **GPS22** will cause the effect of the troposphere to cancel out when the weather conditions over the differenced stations are identical.

- 5). a wide-lane float solution is run using **GPS22**.
- 6). a L1 float solution is run with **GPS22**.
- 7). a L2 float solution is run with **GPS22**. In all four columns of ambiguities in the INTGR file are filled now.
- 8). a **BIAS** program is run. The **BIAS** program reads the following files: *HD.DAT, INTGR, and GPS22.INP. The **BIAS** program works as follows. The float integers found in the L1 and L2 columns are rounded off to the nearest full or half cycle inside the **BIAS** program. The L1 integers are searched to within 2 whole cycles on either side of the rounded L1 value. The L2 integers are searched to within 4 half cycles on either side of the rounded L2 value (if the **BIAS** program senses that a squares the L2 signal is being used, it will allow L2 integers that end in .50). For each possible combination of these "full" L1 and "half" L2 integers a "computed" ion-free and wide-lane ambiguity is calculated. These are differenced from the "observed" ion-free and wide-lane ambiguities found in the INTGR file. The differences for each combination of L1 and L2 integers are then listed in a table (which is shown on the screen when SEARCH is selected from the menu in the **BIAS** program). The pair with smallest difference will point to the best L1 and L2 integer. Choosing ACCEPT from the submenu beneath the table will cause the best L1 integer and best L2

integer to be written to the INTGR file for that satellite and station. Choosing REJECT will keep the original float L1 and L2 integers in the INTGR file. Usually the **BIAS** program will point out the correct integers unless there are only a few minutes of data for that satellite. Fixing the L1 and L2 integer values for the first reference satellite must be avoided because these integers will always be 0.00 for L1 and 0.00 for L2 (the PRN number of the first reference satellite is listed in the **GPS22** menu). If only P-code receivers are used, with full cycles in L2, there will only be full cycle L2 integers (nothing ending in .50).

9). The **BIAS** program will step through the INTGR file one line at a time, doing a search for each line at the user's request. Once all the best L1 and L2 integer choices have been written to the INTGR file, the **GPS22** menu is accessed and the unknown stations are set from "FID" to "SLV" using the STATION LIST submenu (the opposite of STEP 3). Then an iono-free (fixed) solution is run to insure that all integers which were fixed, are indeed the correct integers.

10). The individual RMS values are examined for each station/satellite in the SAVIT file to see if any station/satellite RMS has increased dramatically (more than .02 meters). Also the post-fit double difference residual plots (PFR***.PLT) are examined to see if any of the satellites seem to "separate" from the other satellite residuals -- this is

another indication of a "bad fix". If problems are seen in the RMS values or in the plots, then the corresponding integers are unfixed by editing the INTGR file with a text editor (add .01 to the fixed integers). In unfixing any integer, both L1 and L2 are unfixed. Then the ion-free solution is rerun and the RMS values and the plots are reexamined. This is done until the RMS values and the plots indicate that all integers that can be fixed have been fixed correctly. **BIAS** need not be run again. Most, if not all, of the integers can be fixed using this method, especially for baselines under 100 km.

4.7 Adjustment of HARN Project

After HARN GPS data pro_processing, all vectors between GPS stations are obtained. If the RMS is bigger than requested accuracy than some stations may need to be reobserved. After reobservation, the relative vectors need to be reprocessed.

In order to achieve a more homogenous GPS benchmark data set, the A-Order, B-Order, and some selected CORS stations constituting the Iowa HARN project were combined in a special adjustment. While this was not a minimally constrained adjustment, the CORS coordinates were considered to be so accurate that they

would not bias the adjustment, which will be assumed as a fixed station.

The large variance of unit weight is consistent with those obtained by other GPS network adjustments, and generally indicative of overly optimistic assessments of GPS vector accuracies. Estimates of ellipsoidal horizontal coordinates and height accuracy relative to the CORS stations are obtained by formal error propagation in the adjustment program. In the adjustment, a priori standard deviations of adjusted ellipsoid horizontal coordinates and heights are given from the results of OMNI GPS vector data process for the 181 points. However, it is more appropriate to consider a posteriori statistics. So in the adjustment, a posteriori standard deviation of unit weight will be estimated in the adjustment at the same time to improve the adjustment results. This is because the statistics are typically obtained by simple scaling of the vector covariance matrix, which is expressed in an X,Y,Z system. It is known that GPS measurements are less accurate in height than in latitude or longitude; usually a factor of 2 to 3 times less accurate. But, a priori variances and covariances assigned to vectors by GPS reduction software might not accurately reflect this ratio. In such circumstances, it is not appropriate to scale the horizontal and the vertical components of GPS error by the same

amount. Instead, separate variance components for scaling of horizontal and vertical error should be estimated. For this reason, the a posteriori height accuracy of a few cms is not considered completely definitive at this point. It is useful to consider the GPS vector adjustment residuals.

It must be emphasized that residuals from a least-squares adjustment underestimate the true measurement error. The GPS stations were established in a dual occupation scheme; in general, each ellipsoidal height is determined from only two measurements. A more realistic assessment of the actual error of individual vectors would be roughly 40% larger than the amount indicated by the residual statistics, or 2-3 cm (one-sigma) of random error in ellipsoid height determinations. The situation is alleviated somewhat by the fact that we are interested in adjusted ellipsoidal heights. If one assumes the absence of systematic errors, the adjusted values should be more accurate than the individual determinations by a compensating 40%. Although this figure is probably optimistic, the adjusted ellipsoid heights might be considered to have 1-2 cm (one-sigma) accuracy with respect to the CORS stations.

When considering the average discrepancy of 1.0 cm, it must be realized that the ellipsoid heights from the special adjustment, as a

group, are subject to any systematic errors on the baselines connecting the Iowa HARN to the CORS points. From the adjustment and analysis results, additional investigation will be performed on the large residuals associated with the Iowa HARN GPS adjustment. Such investigation may identify problem vectors that should be re-reduced or reobserved. The use of the HARN and CORS Stations with the new methods from the research should allow surveyors to determine precise and accurate orthometric heights.

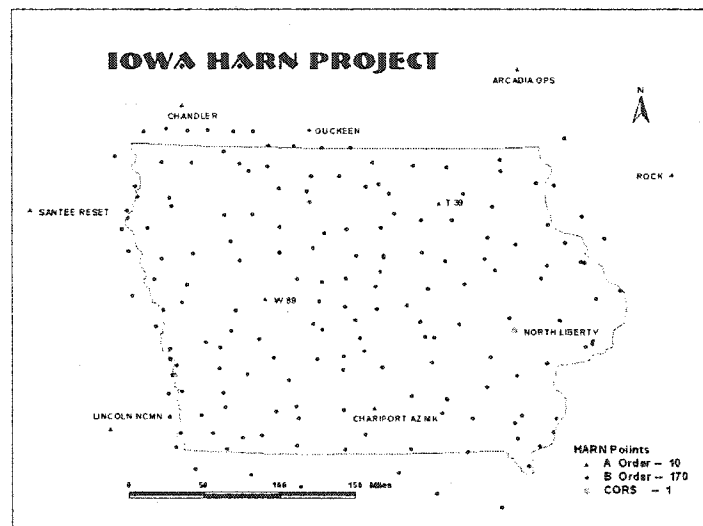


Fig.4.8. Iowa HARN Project

There are total 167 stations and 7 of them will be fixed in the adjustment [Fig. 4.8]. The 7 fixed stations are all A_order station in

Iowa and surround states. The adjustment parameters are show in below table:

PARAMETERS	OBSERVATIONS
Description	Number
All Stations	167
Fixed Stations	7
Distances	861
Free 3-D Stations	160
Coord. Parameters	480
3-D Coord. Diff.	2583
Total Parameters	480
Total Observations	3444
Degrees of Freedom =	2964

The position accuracy is in table 4.1. The average horizontal accuracy of the ellipsoid major axis is 0.0196, and minor axis is 0.0097. We can see major axis from fig.4.9. Most of observations are very good, within the accuracy requested. But some of observations are not good, such as 0.0733 at ARDZ.

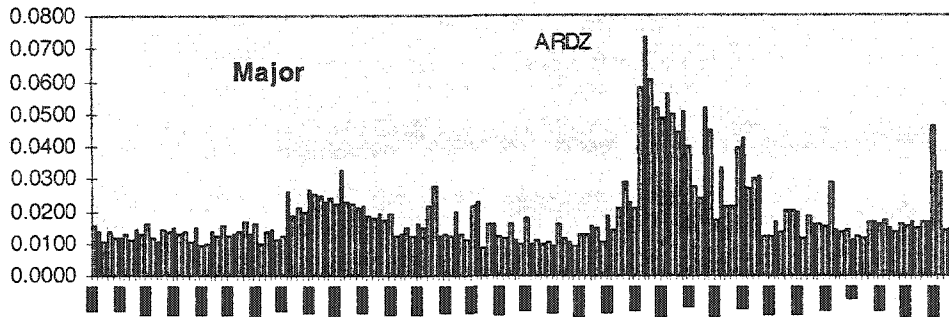


Fig. 4.9 Major of axis of HARN Project

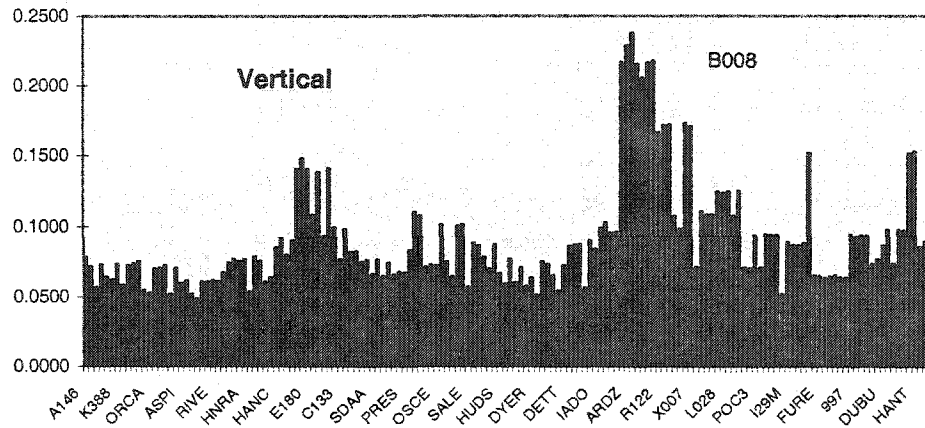


Fig. 4.10 Vertical Accuracy of HARN Project

We can find the difference between horizontal accuracy and vertical accuracy from vertical residuals (see fig.4.10). The vertical accuracy is much lower than the horizontal accuracy.

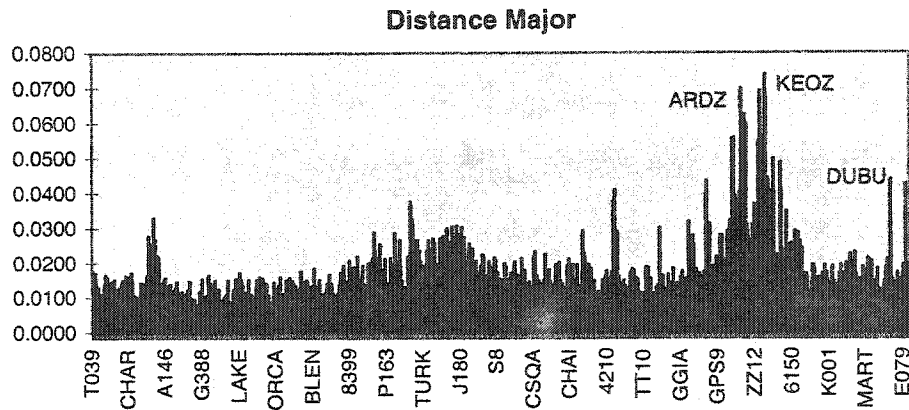


Fig. 4.11 Relative Accuracy of All Stations

From Fig. 4.11, showing the relative accuracy of all stations, we can see the absolute accuracy between all station and the vectors. Because the length of vector is different, the relative precision for all vectors will be more reliable and reasonable for accuracy estimation. The results are shown on the Fig.4.12. The average relative precision is better than 0.5 ppm, and the two worst vectors are BUBU and DVNA.

As we described, GPS is a 3D position system. It should have same accuracy in vertical and horizontal directions. But as we can see from table 4.1 and Fig.4.9 and Fig. 4.10, the vertical accuracy is much lower than we expected. The main reason for this is the geoid model is not satisfactory for the transformation accuracy between two systems.

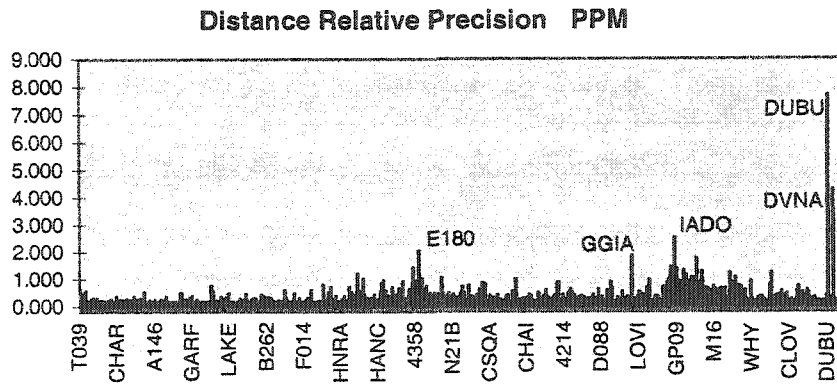


Fig.4.12 Distance Relative Precision of All Stations

In order to improve the vertical accuracy and match to the horizontal accuracy, we have to develop and improve Iowa HRARN project's geoid model. The new geoid model only could improve the horizontal accuracy of HARN; it will also offer a more accuracy model to allow surveyors to convert GPS heights to geodetic heights to meet engineering application requirements.

Table 4.1 Position Accuracy of HARN Project

Station	MAJOR	MINOR	AZ(MAJ)	VERTICAL
A146	0.0152	0.0091	169.82	0.0782
D154	0.0137	0.0078	5.13	0.0713
DIVI	0.0104	0.0076	6.16	0.057

Table 4.1 (continued)

G388	0.0134	0.0096	15.74	0.0727
GARF	0.012	0.0085	11.58	0.0643
HOMA	0.0116	0.0081	14.38	0.0625
K388	0.013	0.0099	7.31	0.0731
LAKE	0.011	0.0086	2.42	0.0588
MANL	0.0141	0.0091	17.22	0.0722
OTGA	0.0131	0.01	175.75	0.0735
SANT	0.0157	0.0125	97.80	0.0752
SLOA	0.0115	0.0071	173.84	0.0549
ORCA	0.0106	0.0072	12.14	0.0532
1600	0.014	0.0095	4.64	0.07
B262	0.0136	0.0107	9.69	0.0704
CARL	0.0148	0.0123	47.12	0.0722
F248	0.0129	0.0033	177.33	0.0524
GP48	0.0136	0.0099	172.57	0.0705
ASPI	0.0103	0.0036	172.26	0.0602
BLN	0.0146	0.0034	170.78	0.0613
EARL	0.0091	0.0053	0.2	0.0522
F014	0.0098	0.0052	11.95	0.0489
J159	0.0134	0.0029	175.11	0.0609
MAPP	0.0121	0.0039	1.03	0.061
RIVE	0.0151	0.0064	143.93	0.0617
TJEB	0.0124	0.0052	160.25	0.0615

Table 4.1 (continued)

AITK	0.0132	0.0064	177.83	0.0672
8399	0.0138	0.0052	163.93	0.0743
D3	0.0164	0.005	162.5	0.077
ET1H	0.013	0.0065	158.71	0.0755
HNRA	0.0157	0.0061	158.39	0.077
P163	0.0099	0.0064	170.3	0.054
PERR	0.0135	0.0061	5.11	0.0788
U88	0.0141	0.0036	178.97	0.0755
X003	0.0112	0.0052	7.93	0.0609
A162	0.0123	0.0066	174.13	0.0641
HANC	0.0261	0.0074	172.97	0.0852
TURK	0.0186	0.0088	173.69	0.0918
U148	0.021	0.0079	156.55	0.0803
W100	0.0196	0.0065	174.08	0.0903
EE15	0.0266	0.0106	166.78	0.1404
4358	0.0252	0.0236	158.07	0.1478
E180	0.0247	0.0196	150.85	0.1406
J180	0.0226	0.0095	160.86	0.1086
J429	0.0242	0.0175	166.13	0.1382
REDP	0.022	0.0089	164.4	0.0936
T150	0.0329	0.0166	136.49	0.1411
C20	0.0228	0.0124	15.37	0.0995
C133	0.0223	0.0051	177.15	0.0769

Table 4.1 (continued)

C342	0.0212	0.0109	161.33	0.098
N21B	0.0213	0.0064	167.39	0.0819
S8	0.0185	0.0079	176.37	0.0826
S106	0.0178	0.0057	176.74	0.0753
SCHE	0.0189	0.0058	176.26	0.0761
SDAA	0.0174	0.0054	174.8	0.0664
SPAN	0.0193	0.0051	172.89	0.0764
CSQA	0.0125	0.0066	171.93	0.0652
ET2M	0.0131	0.0067	174.23	0.0744
GE-0	0.0148	0.0073	139.66	0.066
K105	0.0126	0.0068	170.69	0.0679
PRES	0.0159	0.0065	170.49	0.0674
R104	0.0146	0.0092	174.72	0.0834
158R	0.0214	0.0112	166.36	0.1103
MILL	0.0277	0.01	145.87	0.1077
20AD	0.0125	0.0065	166.58	0.0716
CHAI	0.013	0.0067	171.33	0.0727
OSCE	0.0121	0.0069	170.23	0.0725
PRE2	0.0194	0.0073	169.43	0.101
X103	0.013	0.0074	171.19	0.0749
FAIR	0.0111	0.0065	161.17	0.0645
PT01	0.0216	0.0112	28.57	0.1005
X7	0.0226	0.0075	5.58	0.1014

Table 4.1 (continued)

SALE	0.0089	0.0078	147.99	0.0572
4210	0.0159	0.0084	169.5	0.0884
4214	0.0159	0.0081	1.06	0.0865
BEST	0.0123	0.0115	141.65	0.0781
BROO	0.0116	0.0081	163.39	0.0701
HUBB	0.0157	0.0084	165.93	0.0868
HUDS	0.011	0.0073	172.26	0.0667
TT10	0.0098	0.0079	149.17	0.0597
C037	0.0178	0.007	150.38	0.077
D088	0.0096	0.0082	149	0.0604
PRAT	0.011	0.0083	171.84	0.071
STO5	0.0099	0.0074	167.57	0.0578
DYER	0.0102	0.0065	163.4	0.0637
GGIA	0.0092	0.0057	157.41	0.0517
HART	0.0159	0.0072	164.02	0.0749
L87	0.0118	0.008	165.28	0.0731
LOVI	0.0106	0.0082	173.2	0.0655
IIBA	0.0093	0.0068	19.49	0.0545
DETT	0.013	0.0094	47.8	0.0725
CHAZ	0.013	0.009	175.86	0.0861
GPS9	0.0151	0.007	169.62	0.0869
GPS1	0.0147	0.0072	167.93	0.0874
IKVA	0.0102	0.0059	178.93	0.0568

Table 4.1 (continued)

LARS	0.0186	0.0064	171.36	0.09
IADO	0.0144	0.0086	166.23	0.0844
STO1	0.0208	0.0074	166.39	0.0991
GP09	0.0288	0.0108	167.77	0.1031
GP15	0.0226	0.007	173.33	0.0958
ST11	0.0212	0.0094	152.47	0.096
39RW	0.0577	0.0242	173.16	0.2169
ARDZ	0.0733	0.0478	120.43	0.229
B008	0.0606	0.0505	61.89	0.2375
MADI	0.0519	0.0211	6.5	0.2157
MPZA	0.0484	0.0268	1.44	0.206
ZZ12	0.0563	0.0206	179.37	0.2166
JACK	0.0496	0.0241	3.36	0.2179
R122	0.0446	0.0175	3.95	0.1668
KEOZ	0.0507	0.0171	18.43	0.1719
M16	0.0398	0.0296	138.95	0.1723
COPE	0.0276	0.0128	11.9	0.1077
HOSK	0.0243	0.0138	26.68	0.0984
PT-0	0.0517	0.021	162.9	0.1733
X007	0.0449	0.0173	178.69	0.1706
PARK	0.0172	0.0046	179.95	0.0716
6150	0.0331	0.008	165.33	0.1109
STAB	0.0218	0.0087	174.61	0.1086

Table 4.1 (continued)

WAPE	0.0218	0.0086	174.6	0.1086
R031	0.0395	0.0137	157.89	0.1246
L028	0.0426	0.0121	173.49	0.1235
BEAN	0.0273	0.0153	20.2	0.1247
A029	0.0301	0.012	171.1	0.1078
2140	0.0309	0.0151	153.06	0.1251
WHYN	0.0125	0.0104	179.95	0.0708
STOR	0.0126	0.0088	170.29	0.0705
POC3	0.0168	0.009	1.67	0.0935
K22R	0.0134	0.0073	0.67	0.0709
K001	0.0202	0.0095	163.51	0.0941
DAL2	0.0202	0.0084	171.78	0.094
ALGO	0.0196	0.0093	167.12	0.094
MILP	0.0115	0.0045	158.1	0.0522
I29M	0.0186	0.0085	13.07	0.0886
46-3	0.0157	0.0087	8.25	0.087
4838	0.0162	0.0083	177.58	0.0867
U016	0.0154	0.0094	177.01	0.0884
E9JA	0.029	0.0094	174.72	0.1515
CLOV	0.0143	0.0065	21.13	0.0657
FURE	0.0138	0.0039	6.7	0.0648
JACP	0.0141	0.0028	178.7	0.0637
K1	0.0113	0.0077	15.65	0.0647

Table 4.1 (continued)

M019	0.0123	0.006	163	0.0656
MART	0.0115	0.0043	168.67	0.064
S388	0.0165	0.0028	178.9	0.0635
997	0.0165	0.0071	10.74	0.0937
C012	0.0162	0.0047	5.41	0.0924
MAPL	0.017	0.0119	59.38	0.0934
T74R	0.015	0.0084	17.77	0.0935
9503	0.0138	0.0044	170.13	0.0739
C175	0.0163	0.0049	169.85	0.0769
DUBU	0.0155	0.0109	174.99	0.0867
DVNA	0.0164	0.0125	176.55	0.0974
E079	0.0145	0.0048	169.17	0.0738
G070	0.0164	0.0119	178.45	0.0974
LANC	0.0165	0.015	161.44	0.0972
CUBA	0.046	0.0174	143.38	0.1515
HANT	0.0318	0.0126	8.02	0.153
E2JS	0.0139	0.0082	172.73	0.0855
E4JS	0.0149	0.0083	5.83	0.0892
G75R	0.0147	0.0079	13.71	0.0824
SUM	3.1304	1.5473		14.6061
Avg.	0.019565	0.009671		0.091288

CHAPTER 5. HARN GEOID HEIGHT MODEL

5.1 Geoid Modeling Theory

The geoid is defined as the gravity equipotential surface that best approximates mean sea level over the entire earth. It has been defined as the datum for the orthometric height system. The irregular shape of the geoid, however, does not allow for an easy computation of the horizontal positions of points. Therefore, a reference surface of regular shape, usually an ellipsoid, is selected to best approximate the geoid either locally or globally.

The geometric relationship between the geoid and the reference ellipsoid surface can be fully described by their separation and the slope of the geoid with respect to the reference ellipsoid (Figure 6.2). The former is known as the geoidal height (N), and the latter is known as the deflection of the vertical.

The deflection of the vertical is defined as the spatial angle between the normal gravity vector and the actual gravity vector. It can be decomposed into two orthogonal components, i.e., the north-south (along the astronomical meridian) and the east-west (the prime vertical) components, denoted by ξ and η , respectively.

The deflections of the vertical can be measured either on the surface of the Earth or on the geoid. A surface deflection of the vertical refers to the spatial angle measured on the surface of the earth between the directions of the plumb line and the normal (through point P) to the reference ellipsoid. A geoidal deflection of the vertical is the angle measured on the geoid between the directions of the plumb line and the ellipsoidal normal (through point P) to the reference ellipsoid (Figure 6.2).

Some tasks in geodesy require the deflections of the vertical at the geoid while others require the deflections of the vertical at the earth's surface. Differences in deflections of the vertical between the terrain and the geoid are due to the effect of the curvature of the actual plumb line which can be estimated by different methods.

In geodetic applications, knowledge of the geoidal heights and deflections of the vertical are required for the following purposes:

- to reduce geodetic observations made on the earth's surface to the ellipsoid;
- to relate three-dimensional geocentric coordinates to the conventional horizontal positions and orthometric heights;
- to facilitate coordinate transformation between the geodetic coordinates and the actual surveying (astronomical) coordinates.

To date, various approaches have been proposed by geodesists to determine the geoidal heights and deflections of the vertical. The primary approaches include the gravimetric method, the astro-geodetic method, the astro-gravimetric method, satellite dynamic solution for potential coefficients, and direct determination from 3-D geocentric coordinates and orthometric heights (see Vanicek and Krakiwsky 1986; Moritz 1989; and Torge 1980).

The gravimetric method calculates the geoidal heights and the deflections of the vertical from gravity anomalies using the well-known Stokes and Veiling-Meitiesz integration formulas, which should be carried out over the entire surface of the earth.

The gravimetric deflection of the vertical thus obtained is the angle between the actual plumb line and the normal to a geocentric reference ellipsoid, measured at the geoid (geoidal deflection).

The astro-geodetic method calculates the surface deflections of the vertical from observed astronomical coordinates. The obtained values are then used to calculate the geoidal height difference between adjacent stations through astro-geodetic leveling.

The astro-gravimetric method utilizes both the gravimetric and astro-geodetic methods. It interpolates the surface deflections of the vertical by combining the observed surface deflections of the vertical and its modified gravimetric deflections of the vertical obtained from the Vening-Meinesz integration which is carried out only over a confined area or region. These values are then used to calculate the geoidal heights.

There are advantages and disadvantages in each of the above approaches. For instance, the gravimetric solution provides good local detail, but the long wavelength feature is biased because of the irregular distribution of gravity observations on the surface of the earth. The satellite solution has unbiased and homogeneous long wavelength features, but lacks sufficient and adequate detail. The accuracy of astrogeodetic solutions deteriorates as the distance from the initial point of the geodetic horizontal network increases. Therefore, whenever possible, the combination of different kinds of data

is should be used in order to increase the quality of the results (see Vanicek and Krakiwsky 1986; Moritz 1989).

5.2 The National Geoid Height Models

The GEOID96 and G96SSS geoid height models were published by the National Geodetic Survey on October 10, 1996 (see Fig.1.3 GEOID96 Model for the United States). These two models were released in a strategy to accommodate the disparate requirements of the surveying and mapping community and the scientific community. The G96SSS model is a purely gravimetric model. It is geocentric, relative to the GRS80 ellipsoid, and is referenced to the ITRF94 (1996.0) frame. G96SSS is suited for various investigations, such as relating GPS receivers mounted in buoys to ocean circulation models.

GEOID96, on the other hand, was developed to support direct conversion between ellipsoid heights expressed in the NAD 83 (86) reference frame and orthometric heights expressed in the NAVD 88 vertical datum. Due to the non-geocentricity of the NAD 83 (86), coupled with a probable constant bias in the definition of NAVD 88, we will find discrepancies of a meter or more between a geocentric geoid

model, such as G96SSS or GEOID96, and GPS on leveled benchmark values. By means of known reference frame relationships, and 2951 GPS benchmark points, a smooth conversion surface was developed to relate the G96SSS model and the GEOID96 model. Therefore, G96SSS and GEOID96 share identical high precision, high-resolution characteristics, and differ only in broader characteristics. Details on the motivation and computational techniques underlying this pair of geoid models can be found in Milbert (1995), Milbert and Smith (1996), Smith and Milbert (1997a), and Smith and Milbert (1997b).

5.3 Iowa HARN GEOID Model

The problem in HARN project geoid model (Fig. 5.1, Iowa GEOID96 Model) is that the GEOID96 model is a national model, which has the best fit for large area but giving a lower accuracy. It is not meant for local regions requiring a high accuracy. For Iowa, at state level, an accuracy of 15 to 30 cm obtained using the GEOID96 is not good enough for engineering applications.

A GPS elevation data on leveled benchmark is prepared from Iowa HARN project adjustment. The data set has adjusted NAD 83

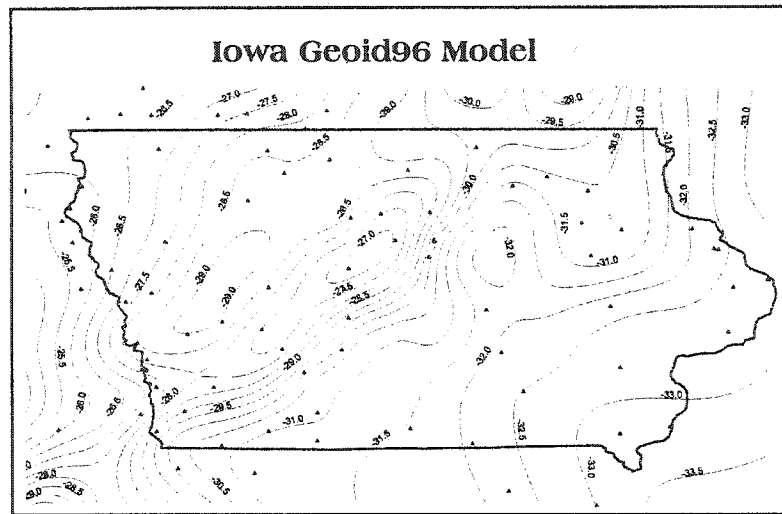


Fig. 5.1 Iowa GEOID96 Model

ellipsoid heights measured by A-Order, B-order, CORS GPS specifications, and adjusted NAVD 88 orthometric heights that are of first order, second order, and third order. It contains 81 GPS benchmarks in and near Iowa (see Fig.5.2 Iowa Leveling Points).

There are total 81 leveled benchmark points in the Iowa HARN, which have GPS elevation from HRAN and M.S.L from geodetic leveling. Using GEOID96, we could convert all GPS heights to orthometric heights. The position, M.S.L elevation (elevation determined by geodetic leveling), GPS heights (ellipsoid height), geoid undulation, orthometric

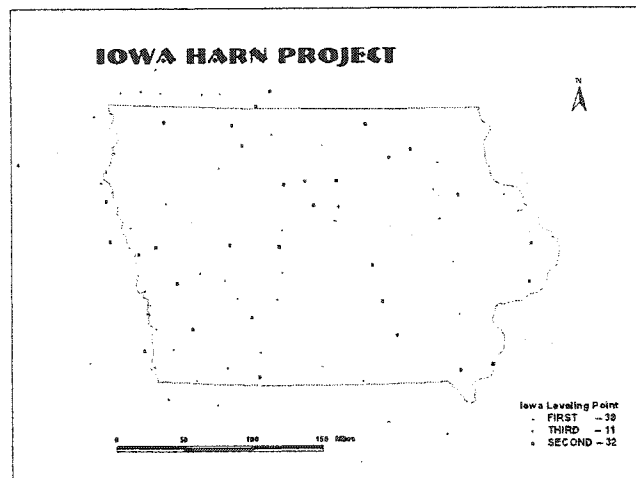


Fig.5.2 Iowa Leveling Points

and leveling order, the difference between orthometric heights converted GPS heights and M.S.L heights are shown in table 5.1.

Table 5.1. The Difference between GPS Height and Orthometric Height

PID	Latitude	Longitude	M.S.L Elev	ELLIP_h	GEOID_h	V_ORDER	Differ_H
OP0929	43.67362	-94.24616	342.891	314.69	-28.16	SECOND	0.041
NJ0606	42.57940	-91.54692	304.112	273.56	-30.55	SECOND	0.002
NK0459	42.72799	-93.74822	353.678	326.17	-27.54	SECOND	-0.032
OP1351	43.51024	-94.44776	369.056	340.86	-28.15	SECOND	0.046
NK0427	42.48335	-93.61755	372.625	345.58	-27.06	SECOND	-0.015
NK0186	42.73971	-93.30085	368.954	340.57	-28.39	SECOND	-0.006
MG0328	41.64573	-90.58086	237.776	205.03	-32.69	SECOND	0.056

Table 5.1 (continued)

NL0104	42.03764	-94.08670	318.262	290.97	-27.27	SECOND	0.022
NK0179	42.98021	-92.53943	299.264	268.08	-31.10	SECOND	0.084
NJ0232	42.05108	-90.53431	269.781	237.11	-32.66	SECOND	0.011
OO0274	43.33467	-92.87812	363.458	333.75	-29.65	SECOND	0.058
OO0226	43.06421	-92.22451	351.765	321.13	-30.58	SECOND	0.055
LD0531	40.78039	-91.11734	210.193	177.03	-33.19	SECOND	-0.027
LD0612	40.72151	-91.56385	213.467	180.31	-33.19	SECOND	-0.033
OP0140	43.10264	-94.63957	372.810	344.21	-28.57	SECOND	0.030
NL0062	42.69062	-94.03006	346.060	317.60	-28.36	SECOND	0.100
MH0109	41.46303	-92.64701	263.407	231.27	-32.12	SECOND	0.017
OP0152	43.31058	-94.79258	387.789	359.31	-28.43	SECOND	0.049
MH0053	41.10643	-92.44496	256.504	224.01	-32.50	SECOND	-0.006
OP0559	43.32431	-95.78303	442.886	416.10	-26.77	SECOND	0.016
MK0194	41.92338	-96.08942	315.714	288.11	-27.29	SECOND	0.314
MH0163	41.84856	-92.78302	314.070	282.27	-31.72	SECOND	0.080
NM0663	42.47109	-96.57185	394.155	368.39	-25.74	SECOND	0.025
NL0186	42.05552	-94.78369	366.039	337.60	-28.34	SECOND	0.099
NM0544	42.04647	-96.49643	423.116	397.39	-25.75	SECOND	-0.024
NL0444	42.00684	-95.84938	413.480	385.60	-27.83	SECOND	0.050
MJ0521	41.14621	-95.28902	388.514	360.27	-28.25	SECOND	-0.006
LF1108	40.90034	-95.95920	385.357	358.05	-27.34	SECOND	-0.033
MJ0955	41.63098	-95.52626	361.529	332.23	-28.99	SECOND	0.309
NM1382	42.80819	-97.85983	512.591	486.77	-25.84	SECOND	-0.019

Table 5.1 (continued)

LF0049	40.65360	-94.34499	348.969	317.74	-31.25	SECOND	-0.021
MJ0136	41.28249	-94.46282	416.462	386.50	-29.86	SECOND	0.102
NJ1003	42.34255	-91.82096	286.800	255.97	-30.80	THIRD	0.030
LF1531	40.92672	-95.55660	300.700	271.70	-28.97	THIRD	0.030
NJ1008	42.64158	-91.90821	306.200	275.48	-30.67	THIRD	0.050
LE0532	40.76809	-93.48721	338.280	306.75	-31.53	THIRD	0.000
NJ0893	42.39613	-90.66571	358.640	326.25	-32.38	THIRD	0.010
LF1488	40.74512	-94.78994	376.600	345.65	-30.95	THIRD	0.000
LF0026	40.91402	-94.33737	378.300	347.41	-30.91	THIRD	-0.020
NL0235	42.85160	-94.97588	421.400	392.63	-28.67	THIRD	0.100
MJ1465	41.67439	-94.84512	425.000	397.48	-27.48	THIRD	0.040
MJ0283	41.74478	-95.20908	431.150	402.20	-28.90	THIRD	0.050
AA5717	43.60921	-96.43244	439.130	412.77	-26.36	THIRD	0.000
NJ0104	42.11668	-90.22004	191.175	158.17	-32.92	FIRST	0.085
NL0066	42.22493	-94.04791	345.712	318.84	-26.86	FIRST	0.012
OP0124	43.22645	-94.22135	351.223	322.50	-28.64	FIRST	0.083
MG0566	41.88330	-91.64675	247.108	214.77	-32.23	FIRST	0.108
MG0476	41.32619	-91.56100	229.568	196.79	-32.75	FIRST	0.028
NM0034	42.98600	-96.48546	358.616	332.68	-25.90	FIRST	0.036
NM0820	42.78239	-96.59274	347.196	321.55	-25.64	FIRST	0.006
NM1955	42.66926	-96.66811	340.413	314.79	-25.59	FIRST	0.033
NJ0599	42.93541	-91.84981	359.629	329.12	-30.54	FIRST	-0.031
OP1258	43.64709	-94.98223	440.209	412.67	-27.52	FIRST	0.019
NJ0258	42.40285	-90.70898	322.104	289.77	-32.33	FIRST	0.004

Table 5.1 (continued)

OO0687	43.12590	-93.50185	385.877	357.09	-28.76	FIRST	0.027
NJ0367	42.58606	-90.90517	341.978	309.89	-32.06	FIRST	0.028
OP1268	43.63773	-95.24503	435.028	407.72	-27.27	FIRST	0.038
NM0113	42.21456	-96.21511	325.803	299.25	-26.49	FIRST	0.063
LD0564	40.06397	-91.77810	213.525	181.07	-32.53	FIRST	-0.075
OQ0977	43.36123	-96.79580	413.001	386.70	-26.30	FIRST	0.001
OQ1084	43.64677	-96.13343	465.683	439.27	-26.39	FIRST	0.023
NK0284	42.48510	-93.26263	343.764	313.91	-29.73	FIRST	0.124
NK0300	42.33213	-93.30981	339.037	309.18	-29.76	FIRST	0.097
MJ1402	41.14914	-95.81236	297.886	270.52	-27.32	FIRST	0.046
NK0285	42.47070	-93.26253	345.801	315.88	-29.78	FIRST	0.141
NL0374	42.47306	-95.72781	367.750	340.26	-27.40	FIRST	0.090
OP1261	43.63709	-95.85120	498.415	471.94	-26.42	FIRST	0.055
MJ0074	41.77699	-94.04840	312.835	283.55	-29.19	FIRST	0.095
OP0609	43.88351	-95.92896	558.661	532.43	-26.23	FIRST	0.001
MJ1382	41.40151	-95.89412	300.733	272.49	-28.18	FIRST	0.063
MJ1360	41.30743	-95.94418	317.986	290.31	-27.65	FIRST	0.026
MJ0984	41.55098	-95.98508	306.140	277.65	-28.45	FIRST	0.040
LF1452	40.40360	-95.61515	271.148	241.03	-30.09	FIRST	0.028
LF1169	40.74374	-95.80981	282.940	253.84	-29.05	FIRST	0.050
MJ0089	41.48958	-94.11105	334.006	303.74	-30.19	FIRST	0.076
LF0231	40.61038	-95.22041	296.766	265.99	-30.78	FIRST	-0.004
LE0188	40.62951	-92.91840	314.572	282.52	-32.09	FIRST	-0.038
LG1229	40.76089	-96.70698	362.065	336.09	-25.92	FIRST	0.055

Table 5.1 (continued)

LE0139	40.19543	-92.58333	299.116	266.87	-32.26	FIRST	-0.014
MJ0181	41.49666	-94.66008	435.981	408.26	-27.66	FIRST	0.061
LF0151	40.36114	-94.91953	333.730	302.52	-31.25	FIRST	-0.040

The statistic analysis results for the difference between GPS orthometric heights and M.S.L heights by geodetic leveled of 81 points are

Table 5.2 Statistic Analysis Results

Moments			
N	81	Sum Wgts	81
Mean	0.037951	Sum	3.074
Std Dev	0.061472	Variance	0.003779
Skewness	2.156923	Kurtosis	8.159071
USS	0.418968	CSS	0.302308
CV	161.9798	Std Mean	0.00683
T:Mean=0	5.556249	Pr> T	0.0001
Num ^= 0	78	Num > 0	60
M(Sign)	21	Pr>= M	0.0001
Sgn Rank	1093	Pr>= S	0.0001

Table 5.2 (continued)

Quantiles (Def=5)			
100% Max	0.314	99%	0.314
75% Q3	0.058	95%	0.108
50% Med	0.03	90%	0.099
25% Q1	0	10%	-0.024
0% Min	-0.075	5%	-0.033
		1%	-0.075
Range	0.389		
Q3-Q1	0.058		
Mode	0.05		

Univariate Procedure

Variable=D_LEVEL

Stem Leaf	#	Boxplot
30 94	2	*
28		
26		
24		
22		
20		
18		
16		
14 1	1	
12 4	1	
10 0028	4	
8 03450579	8	
6 1336	4	
4 001669000055568	15	+-----+
2 23567888000368	14	*-----*
0 00011246012679	14	+-----+
-0 9546664	7	
-2 833217410	9	
-4 0	1	
-6 5	1	
		-----+

Table 5.2 (continued)

Multiply Stem.Leaf by 10**⁻²
Extremes

Lowest	Obs	Highest	Obs
-0.075(1)	0.108(77)
-0.040(2)	0.124(78)
-0.038(3)	0.141(79)
-0.033(5)	0.309(80)
-0.033(4)	0.314(81)

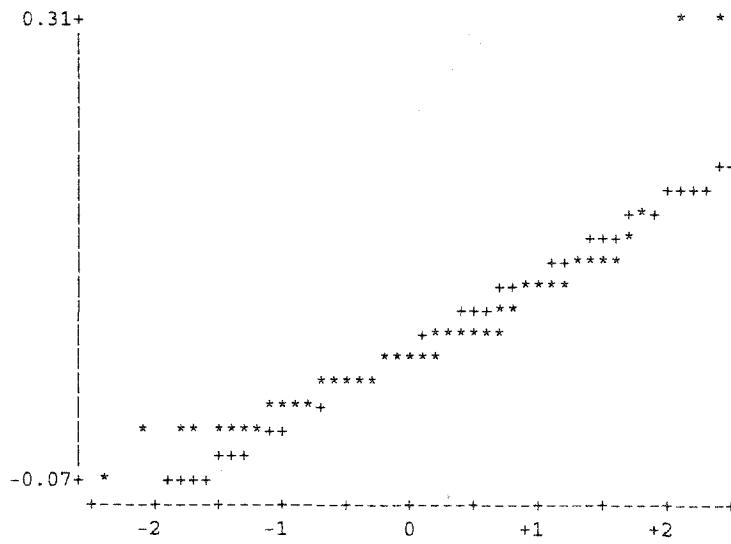


Fig. 5.3 Normal Probability Plot

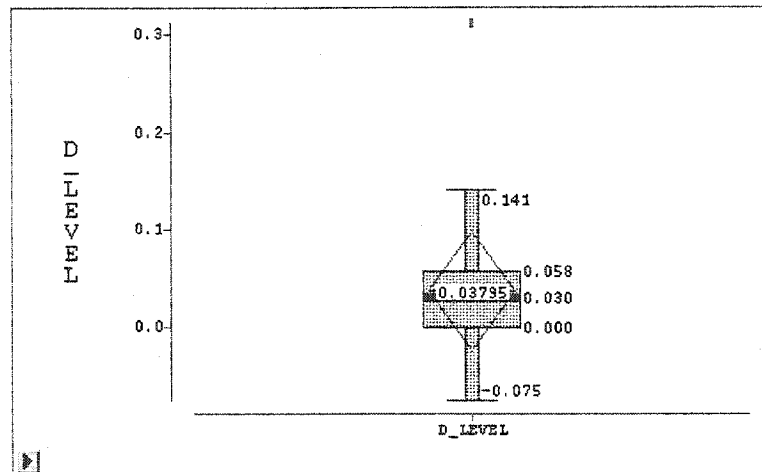


Fig. 5.4 The Differences between GPS Heights and Leveling Heights

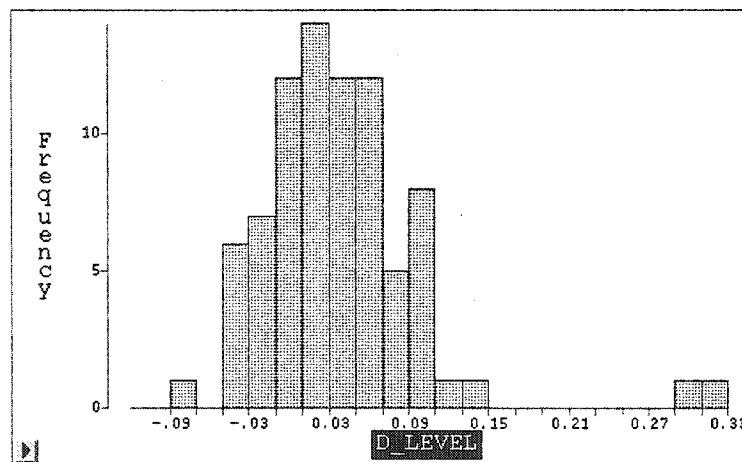


Fig. 5.5 The Difference Frequency Chart

The mean of the differences is 0.038m and standard deviation is 0.061m. The mean of all differences is not zero. This means that there is a 0.038m bias in the Iowa GEOID96 model. We also see that the two differences are significant. The standard deviation of the difference is

0.061m. This standard deviation is too big for engineering applications. From this, we can see that GEOID96 model does not meet engineering application needs in a local area. A better and more accurate local geoid model has to be developed to meet this need.

5.4 A Regional GEOID Model

In this research, a regional geoid model based on the GEOID96 model has been developed. It used the differences between the GPS ellipsoid heights after applying GEOID96 model to GPS heights and the benchmark orthometric heights (M.S.L elevation). Geoid height model values are computed through interpolation from the specified geoid height model grid. In addition, differences between the measured geoid heights and the model geoid heights are accumulated in this research. Because of the possibility of datum offsets in ellipsoidal, orthometric, and/or geoid heights, the second part of this research computes the correction between the Geoid96 and orthometric heights which were measured by first, second and third order geodetic leveling. We are using the following expression of the Geoid height between GPS ellipsoidal height and orthometric height:

$$h = H + N(\text{Geoid96}) + N(\text{Region})$$

where h is ellipsoidal heights from GPS, H is orthometric height from geodetic leveling. The geoid N has two parts, one is the GEOID96 model, and the second is a regional geoid model, which we will develop in this research.

It is reasonable to interpret the geoid correction from the difference between GPS and leveling heights after adding Geoid96. As discussed above, these discrepancies will contain error contributions from GPS and leveling, as well as from the geoid model, itself.

In this section, comparisons are made between geoid models and different sets of GPS ellipsoid heights co-located with orthometric heights computed from differential leveling. It must be understood that in any such comparison, GPS networks are less accurate in determining ellipsoidal height than horizontal position. And, one must be prepared for the possibilities of errors in the level network, particularly if the leveling is not of recent vintage. Isolation of problems in a level network is often hampered by the lack of redundancy, even though the component level lines are double run. For these reasons, the outliers must be detected and eliminated from the modeling data set by checking misclosure of data set. Outliers are detected in part by excessive magnitude, and in part by excessive relative magnitudes when compared to their closest neighbors. The general result that can

be gleaned from this analysis is that the GEOID96 geoid height model and local regional geoid correction model in Iowa will be within its specified error assessment of 10 to 15 cm (one-sigma) over spacing of 50 km or more. Fig. 5.6 shows the height difference between GPS and leveling. From the figure we can see there are two points with biggest difference of 0.316 and 0.310 meters in the west of Iowa, and most area

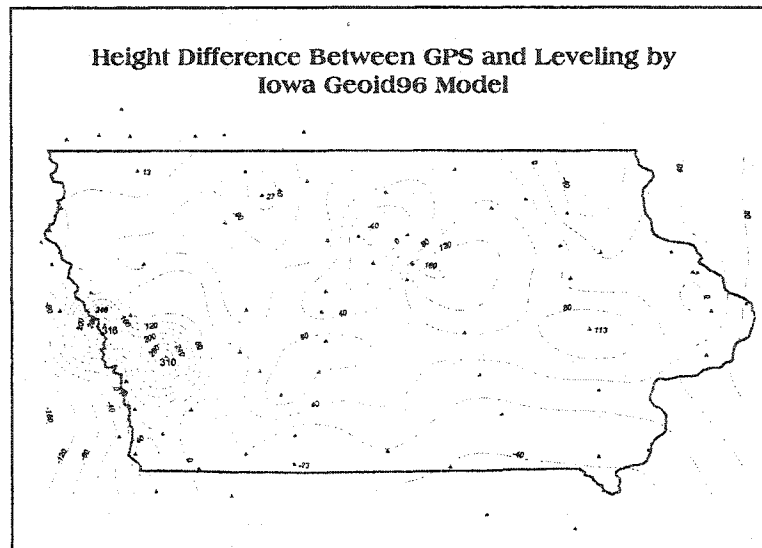


Fig. 5.6 Height Difference between GPS and Leveling

has a difference between 0 to 20 cm. Accordingly, using the differences between the GPS heights after applying GEOID96 and orthometric heights from geodetic leveling we could model the geoid difference in Iowa and use this model to correct GPS height to orthometric heights.

It is important that the relative geoid accuracy in Iowa should be better based upon the new geoid model.

To have the new regional geoid model represent Iowa's geoid correction, we know we should have enough common points with GPS heights and geodetic leveling heights and these common points should cover the whole modeling area.

The figure 5.7 shows the area covered by each point in Iowa. There are total 61 common points in Iowa area. The average area covered by each point is 921.94 square miles. The smallest area is

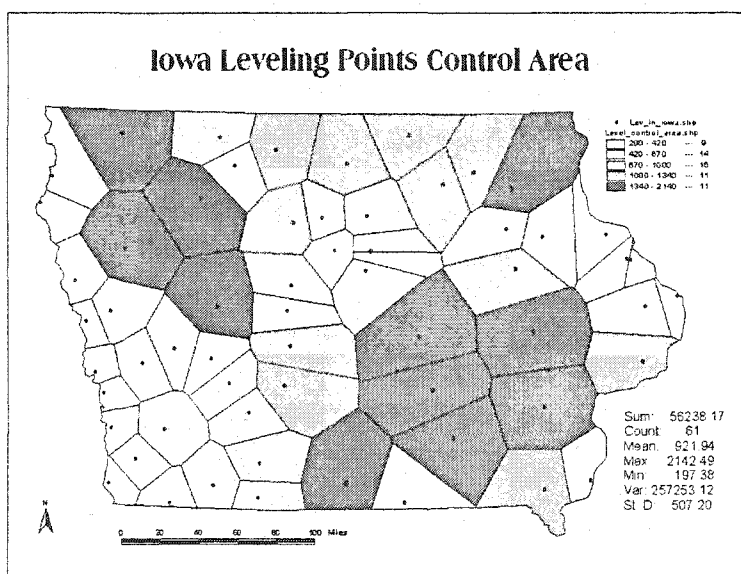


Fig. 5.7 Iowa Leveling Points Control Area

197.38 square miles and the largest area is 2142.49 square miles.

Comparing the Fig. 5.6 and Fig. 5.7 we can find that we have a higher point density in south west area, where we have bigger differences between GPS heights and leveling heights.

The fig. 5.8 shows the leveling point observation orders and its control area. There are 25 first order points, 26 second order points, and only 10 third order points. Obviously, we have a pretty good and vertical leveling network for the new correction geoid model.

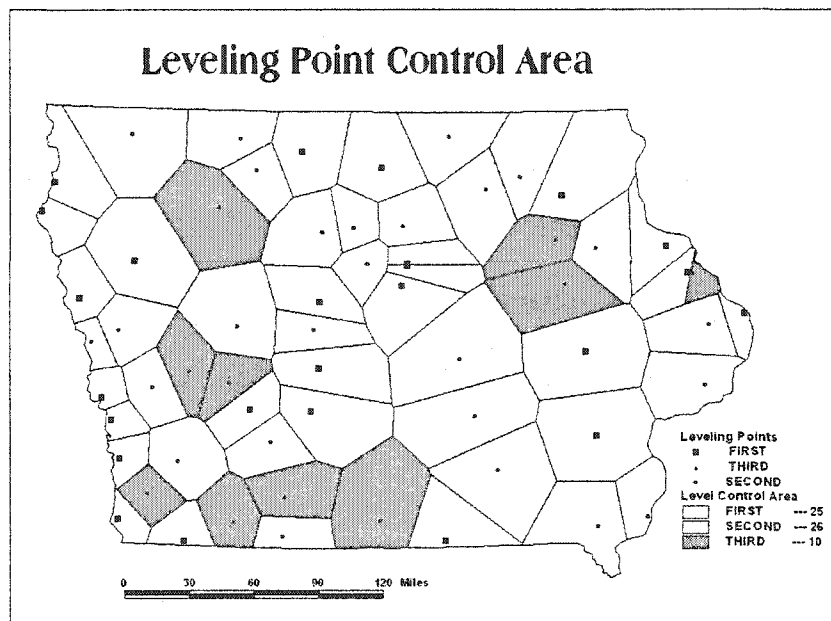


Fig. 5.8 The Leveling Points Order and Control Area

The fig. 5.9 shows the area affected by the height differences between GPS and leveling. The two largest differences affect only a very small area and we have a higher point density in southwest, which will

give us a better geoid correction model in higher geoid change area. Fig.5.10 shows the differences between GPS heights after applying GEOID96 model and the geodetic leveling heights in Iowa and the surround area. To develop a regional geoid model for Iowa, this research not only uses the common points in Iowa, it also uses the points surround Iowa. A total of 79 point in Iowa and the surround area are used in this research.

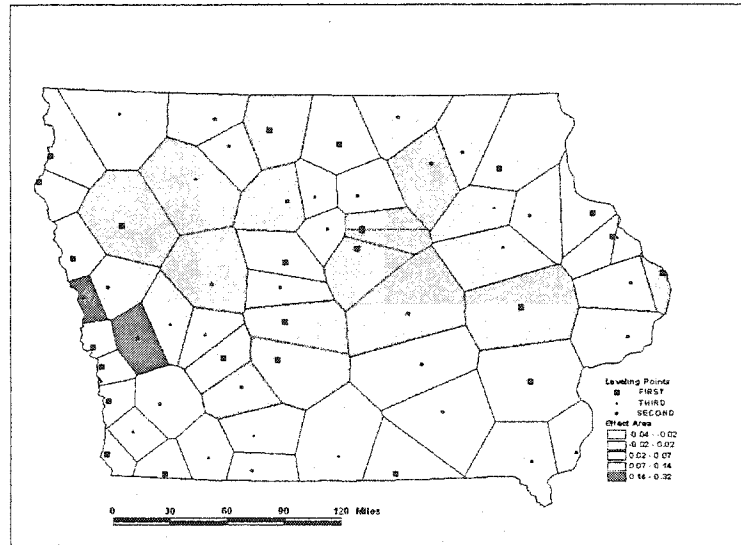


Fig. 5.9 Effect Area of Different Elevations between GPS and Leveling

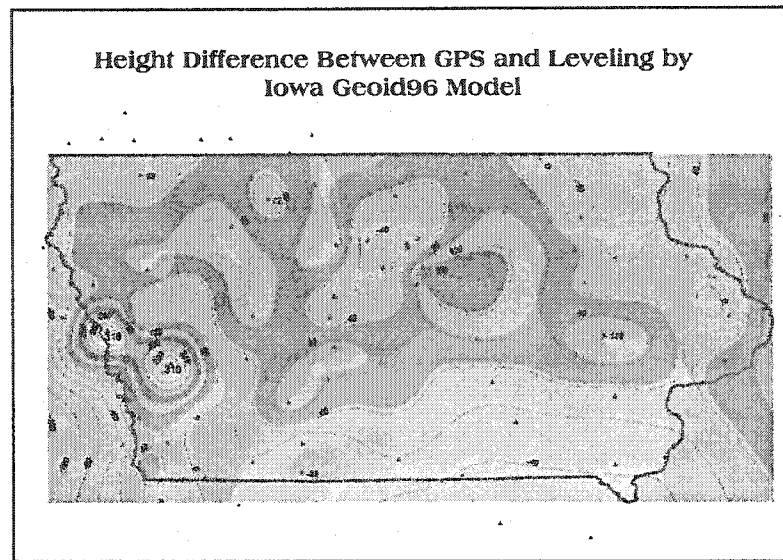


Fig. 5.10 Height Difference Between GPS and Leveling

Table 5.3 GPS Height and Orthometric Height

I	Lat.	Long.	Ellip_h	Geoid	L_h	H_order	Weight	D_GH	D_GHL
1	43.12590	-93.50185	357.09	-28.76	385.877	FIRST	1	-0.031	0.017
2	42.33213	-93.30981	309.18	-29.76	339.037	FIRST	1	-0.099	-0.017
3	42.48510	-93.26263	313.91	-29.73	343.764	FIRST	1	-0.12	-0.042
4	42.47070	-93.26253	315.88	-29.78	345.801	FIRST	1	-0.137	-0.059
5	41.55098	-95.98508	277.65	-28.45	306.140	FIRST	1	-0.045	0.002
6	43.51024	-94.44776	340.86	-28.15	369.056	SECOND	0.4444	-0.049	-0.006
7	41.88330	-91.64675	214.77	-32.23	247.108	FIRST	1	-0.113	-0.051
8	41.74478	-95.20908	402.20	-28.90	431.150	THIRD	0.1111	-0.053	0.007
9	42.57940	-91.54692	273.56	-30.55	304.112	SECOND	0.4444	0.003	0.039
10	42.78239	-96.59274	321.55	-25.64	347.196	FIRST	1	-0.011	0.015
11	41.48958	-94.11105	303.74	-30.19	334.006	FIRST	1	-0.08	-0.026
12	42.58606	-90.90517	309.89	-32.06	341.978	FIRST	1	-0.031	-0.011
13	43.10264	-94.63957	344.21	-28.57	372.810	SECOND	0.4444	-0.027	0.032

Table 5.3 (continued)

14	42.73971	-93.30085	340.57	-28.39	368.954	SECOND	0.4444	0.001	0.069
15	42.47109	-96.57185	368.39	-25.74	394.155	SECOND	0.4444	-0.025	0.013
16	42.69062	-94.03006	317.60	-28.36	346.060	SECOND	0.4444	-0.096	-0.019
17	41.32619	-91.56100	196.79	-32.75	229.568	FIRST	1	-0.024	0.011
18	42.93541	-91.84981	329.12	-30.54	359.629	FIRST	1	0.029	0.04
19	40.74374	-95.80981	253.84	-29.05	282.940	FIRST	1	-0.047	-0.028
20	42.11668	-90.22004	158.17	-32.92	191.175	FIRST	1	-0.087	-0.033
21	40.36114	-94.91953	302.52	-31.25	333.730	FIRST	1	0.042	0.032
22	40.40360	-95.61515	241.03	-30.09	271.148	FIRST	1	-0.023	-0.012
23	43.06421	-92.22451	321.13	-30.58	351.765	SECOND	0.4444	-0.052	-0.04
24	43.88351	-95.92896	532.43	-26.23	558.661	FIRST	1	-0.005	0.025
25	42.98600	-96.48546	332.68	-25.90	358.616	FIRST	1	-0.036	-0.015
26	42.22493	-94.04791	318.84	-26.86	345.712	FIRST	1	-0.013	0.07
27	41.84856	-92.78302	282.27	-31.72	314.070	SECOND	0.4444	-0.083	-0.011
28	42.40285	-90.70898	289.77	-32.33	322.104	FIRST	1	-0.007	0.027
29	41.40151	-95.89412	272.49	-28.18	300.733	FIRST	1	-0.068	-0.026
30	42.05108	-90.53431	237.11	-32.66	269.781	SECOND	0.4444	-0.008	0.047
31	40.92672	-95.55660	271.70	-28.97	300.700	THIRD	0.1111	-0.029	-0.01
32	42.47306	-95.72781	340.26	-27.40	367.750	FIRST	1	-0.086	-0.027
33	42.04647	-96.49643	397.39	-25.75	423.116	SECOND	0.4444	0.025	0.074
34	40.76809	-93.48721	306.75	-31.53	338.280	THIRD	0.1111	0.002	-0.006
35	43.64677	-96.13343	439.27	-26.39	465.683	FIRST	1	-0.025	-0.007
36	41.64573	-90.58086	205.03	-32.69	237.776	SECOND	0.4444	-0.057	-0.001
37	42.34255	-91.82096	255.97	-30.80	286.800	THIRD	0.1111	-0.029	0.029
38	43.67362	-94.24616	314.69	-28.16	342.891	SECOND	0.4444	-0.043	-0.008
39	42.39613	-90.66571	326.25	-32.38	358.640	THIRD	0.1111	-0.007	0.028
40	42.66926	-96.66811	314.79	-25.59	340.413	FIRST	1	-0.036	-0.008
41	43.36123	-96.79580	386.70	-26.30	413.001	FIRST	1	-0.003	-0.007
42	42.00684	-95.84938	385.60	-27.83	413.480	SECOND	0.4444	-0.05	0.008

Table 5.3 (continued)

43	41.14914	-95.81236	270.52	-27.32	297.886	FIRST	1	-0.049	-0.017
44	41.30743	-95.94418	290.31	-27.65	317.986	FIRST	1	-0.026	0.013
45	43.64709	-94.98223	412.67	-27.52	440.209	FIRST	1	-0.023	0.018
46	43.22645	-94.22135	322.50	-28.64	351.223	FIRST	1	-0.087	-0.034
47	40.65360	-94.34499	317.74	-31.25	348.969	SECOND	0.4444	0.023	0.014
48	42.85160	-94.97588	392.63	-28.67	421.400	THIRD	0.1111	-0.1	-0.035
49	43.63709	-95.85120	471.94	-26.42	498.415	FIRST	1	-0.058	-0.031
50	40.06397	-91.77810	181.07	-32.53	213.525	FIRST	1	0.074	0.008
51	42.72799	-93.74822	326.17	-27.54	353.678	SECOND	0.4444	0.031	0.105
52	41.46303	-92.64701	231.27	-32.12	263.407	SECOND	0.4444	-0.021	0.029
53	40.76089	-96.70698	336.09	-25.92	362.065	FIRST	1	-0.057	-0.005
54	40.19543	-92.58333	266.87	-32.26	299.116	FIRST	1	0.014	-0.04
55	43.31058	-94.79258	359.31	-28.43	387.789	SECOND	0.4444	-0.047	0.003
56	43.60921	-96.43244	412.77	-26.36	439.130	THIRD	0.1111	-0.001	0.006
57	43.33467	-92.87812	333.75	-29.65	363.458	SECOND	0.4444	-0.061	-0.049
58	41.49666	-94.66008	408.26	-27.66	435.981	FIRST	1	-0.057	-0.006
59	40.91402	-94.33737	347.41	-30.91	378.300	THIRD	0.1111	0.023	0.032
60	40.72151	-91.56385	180.31	-33.19	213.467	SECOND	0.4444	0.033	0.012
61	42.48335	-93.61755	345.58	-27.06	372.625	SECOND	0.4444	0.018	0.099
62	40.74512	-94.78994	345.65	-30.95	376.600	THIRD	0.1111	0.002	0.003
63	43.63773	-95.24503	407.72	-27.27	435.028	FIRST	1	-0.038	0.001
64	40.61038	-95.22041	265.99	-30.78	296.766	FIRST	1	0.008	0.01
65	42.80819	-97.85983	486.77	-25.84	512.591	SECOND	0.4444	0.015	0.006
66	42.21456	-96.21511	299.25	-26.49	325.803	FIRST	1	-0.06	-0.009
67	40.90034	-95.95920	358.05	-27.34	385.357	SECOND	0.4444	0.03	0.056
68	42.98021	-92.53943	268.08	-31.10	299.264	SECOND	0.4444	-0.086	-0.054
69	42.64158	-91.90821	275.48	-30.67	306.200	THIRD	0.1111	-0.052	-0.011
70	41.14621	-95.28902	360.27	-28.25	388.514	SECOND	0.4444	0.009	0.037
71	43.32431	-95.78303	416.10	-26.77	442.886	SECOND	0.4444	-0.013	0.02

Table 5.3 (continued)

72	42.03764	-94.08670	290.97	-27.27	318.262	SECOND	0.4444	-0.024	0.056
73	41.67439	-94.84512	397.48	-27.48	425.000	THIRD	0.1111	-0.036	0.024
74	42.05552	-94.78369	337.60	-28.34	366.039	SECOND	0.4444	-0.099	-0.024
75	41.10643	-92.44496	224.01	-32.50	256.504	SECOND	0.4444	0.01	0.029
76	41.28249	-94.46282	386.50	-29.86	416.462	SECOND	0.4444	-0.103	-0.066
77	41.77699	-94.04840	283.55	-29.19	312.835	FIRST	1	-0.093	-0.022
78	40.62951	-92.91840	282.52	-32.09	314.572	FIRST	1	0.043	0.019
79	40.78039	-91.11734	177.03	-33.19	210.193	SECOND	0.4444	0.026	0.013

As discussed above, we could use the differences (D_GH in the table 5.3) between GPS heights and leveling heights for all leveling points to establish a regional geoid model for the state of Iowa. In this research we use a polynomial function to describe geoid correction model for Iowa. After comparing different order polynomials and analyzing the geoid physical property in Iowa, we use a third order polynomial function as the new regional geoid correction model for Iowa. The results are:

$$N_c = f(x, y) = \sum_{i,j} a_i x^i y^{n-j} \quad 5.1$$

where (x, y) are the horizontal coordinates of a point in a local coordinate system. n is the order of polynomial.

$$i=0,1,2,3 \text{ and } j=0, 1, 2, 3$$

The differences between GPS heights and geodetic heights, which are functions of position, are used to determine the coefficients of the best fitting polynomial by applying the least squares approximation technique. In this research for the best fit we are also concerned with the order of geodetic leveling. There are three types of leveling order points in this research. The weights we used in the research are listed in the table 5.3. The polynomial coefficients are listed in table 5.4.

Table 5.4. Iowa HARN Geoid Correction Model

a ₀ : -0.08120186	a ₁ : -0.00509963	a ₂ : 0.00605255
a ₃ : 0.00030984	a ₄ : -0.00027834	a ₅ : -0.01986711
a ₆ : 0.00248352	a ₇ : 0.00564225	a ₉ : -0.00044973
a ₁₀ : 0.04703129	a ₁₁ : 0.00916119	a ₁₂ : 0.00000711
a ₁₃ : 0.00128015	a ₁₄ : 0.00016285	a ₁₅ : -0.00681826

The estimated posterior variance factor is 0.029m. If we test the residual for all points, we have:

Total Residue	79
Residue < 1 sigma	53
Residue > 1 sigma	19
Residue > 2 sigma	5
Residue > 3 sigma	2

From the above statistical tests, we can see the fit satisfies the model accuracy requirement. By using this regional geoid model, we can convert all GPS heights to orthometric heights to support all engineering applications and to establish the surveying network. This regional geoid correction model improved vertical accuracy Iowa HARN.

5.5 Model Test

The purpose of creating a geoid correction model is to try to correct new GPS project height to orthometric height by the geoid correction method we developed in HARN project. To test the method and accuracy, we tested the method using six GPS points from three new GPS projects, which are:

- Iowa Hwy. 2 and US 63 project from Centerville east to US 63 south and from Bloomfield north to Ottumwa
- US 30 widening project from the four-lane section east of Carroll to the US 169 interchange at Ogden.
- US 52 from the Jct of IA 386 and J. F. Kennedy North and West to Luxemburg,

the Table 5.5 Region Geoid Model Test Results gives the results

Table 5.5 Region Geoid Model Test Results

ID	Leveling	Geoid96	Elli_Leveling	Diff_h	Correct	New_d
683(G683)	275.779	-31.673	244.180	-0.074	-0.044	-0.118
20RWM1963	233.772	-33.032	200.680	0.060	-0.082	-0.022
X177(G992)	276.117	-32.120	244.044	-0.047	-0.076	-0.123
B89201	229.568	-32.754	196.755	0.059	-0.077	-0.018
J178(G998)	332.781	-31.779	300.960	0.042	-0.073	-0.031
Mean				0.008	-0.070	-0.062
St.d				0.064	0.015	0.053

The model accuracy is beyond the requested accuracy range 1–3 cm \pm 1 sigma. We can see the new geoid model is not satisfactory for Iowa, even though it has improved 10cm-15cm plus 1 sigma accuracy of Geoid96 to 5 to 10 cm. It is still not good enough to meet engineering application needs. The geodetic leveling accuracy requirement is as follows [34]:

	<u>First order</u>	<u>Second order</u>	<u>Third order</u>
Accuracy	5.0mm*sqrt(L)	8.0mm*sqrt(L)	12.0mm*sqrt(L)

Where L = the distance in unit of km between two points. In the projects we tested, L=5km, so we are close to third order leveling accuracy.

To check geoid height changes, we have selected 11 CORS stations within and surround Iowa (Fig. 5.11), and collected GPS observation data for day285 in 2000; day85 and day282 in 2001; and day78 in 2002. Each day we have observation data for 24-hours. We processed each station by using Iowa DOT as the reference point. The position accuracy is better than 1.5 cm. All DGPS results are listed in table 5.6.

From Tables 5.6 and Table 5.7, we can see heights change with time; the biggest change is more than 10 cm and many of them are more than 5 cm, which are much bigger than the DGPS accuracy. The changes are significant. If we try to convert these GPS heights to

Table 5.6 DGPS Results for CORS Stations around Iowa

Point	Lat	Lon	Day78	Day282	Day85	Day258
DOT	42.02224017	93.62223013	263.7419	263.7419	263.7419	263.7419
NLIB	41.77158603	91.57488815	208.7463	208.5648	208.6274	208.5295
BLRW	43.23024816	90.53077188	192.2567	192.2137	192.2256	192.1674
ITHM	39.57595477	94.17014733	263.8725	263.8335	263.8794	263.8127
OMH1	41.77827107	95.91129717	399.2346	399.2799	399.2123	399.2383
RIS1	42.01227556	90.22561251	217.8228	217.7902	217.8355	217.8368
SLAI	41.90057869	93.69886262	291.0198	291.0146	N/A	291.0180
STP1	44.30394371	95.90329811	355.2475	255.2573	N/A	355.2681
WDLM	44.67220753	95.44857228	296.0816	296.0190	296.0714	296.0033
WNCI	39.66460951	90.47674155	142.5617	142.6577	142.5538	142.5861
NLGN	42.20673905	97.79528970	501.8206	501.8434	501.9387	501.8434

orthometric heights, the results would change. Obviously, the orthometric would not match the geodetic leveling height and they would not meet engineering application requirement either. The reason for this is that geoid has changed from the last model. So we can see that no matter which fixed geoid model is used, such as GEOID96 or GEOID99, one would not get the requested accuracy you convert the heights between two systems. From this point, we know we not only need to develop a geoid model for regional, we also need to monitor and describe the geoid changes.

Fig. 5.11 Shows the GPS height contours of Iowa and surround CORS posits as we selected. From the Fig. 5.11 we can see the height

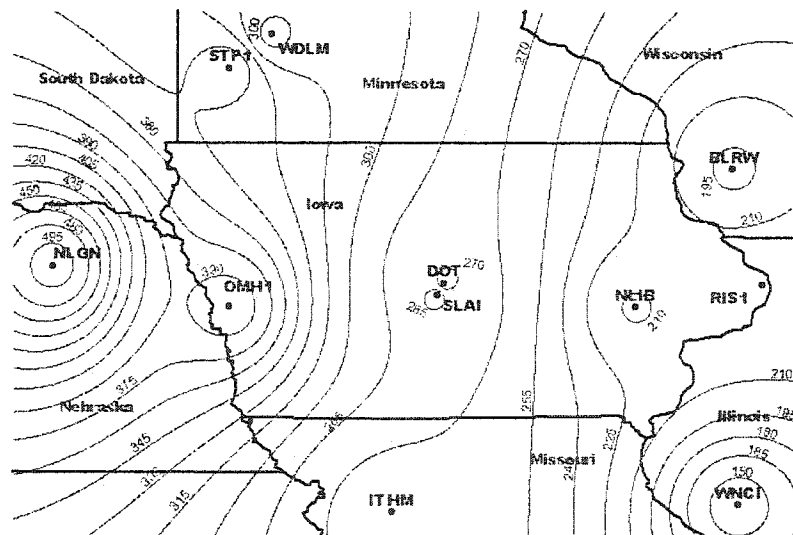


Fig. 5.11 GPS Height Contours of Iowa and Surround CORS Points

Table 5.7 DGPS Height Difference for CORS Station around Iowa

Point	Lat	Lon	Day78	Diff.282-78	Diff.85-282	Diff.258-85
DOT	42.02224017	-93.62223013	263.7419	0.0000	0.0000	0.0000
NLIB	41.77158603	91.57488815	208.7463	-0.1815	0.0626	-0.0979
BLRW	43.23024816	90.53077188	192.2567	-0.0430	0.0119	-0.0582
ITHM	39.57595477	94.17014733	263.8725	-0.0390	0.0459	-0.0667
OMH1	41.77827107	95.91129717	399.2346	0.0453	-0.0676	0.0260
RIS1	42.01227556	90.22561251	217.8228	-0.0326	0.0453	0.0013
SLAI	41.90057869	93.69886262	291.0198	-0.0052	N/A	0.0034
STP1	44.30394371	95.90329811	355.2475	-0.0202	N/A	0.0408
WDLM	44.67220753	95.44857228	296.0816	-0.0626	0.0524	-0.0681
WNCI	39.66460951	90.47674155	142.5617	0.0960	-0.1039	0.0323
NLGN	42.20673905	97.7952897	501.8206	0.0228	0.0953	-0.0953

differences among all point are flat. The height of west is higher than the height of east. The height of north is higher than the height of south. Table 5.7 lists the location and DGPS heights for all points. We

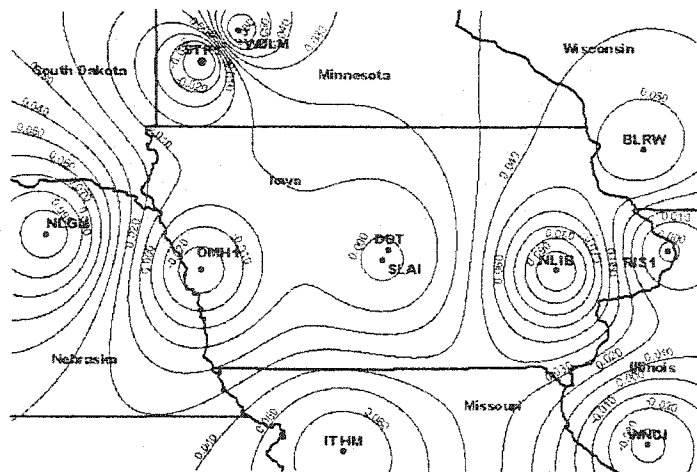


Fig. 5.12 Difference between Day85 and day258

also can see the height differences among different observation days.

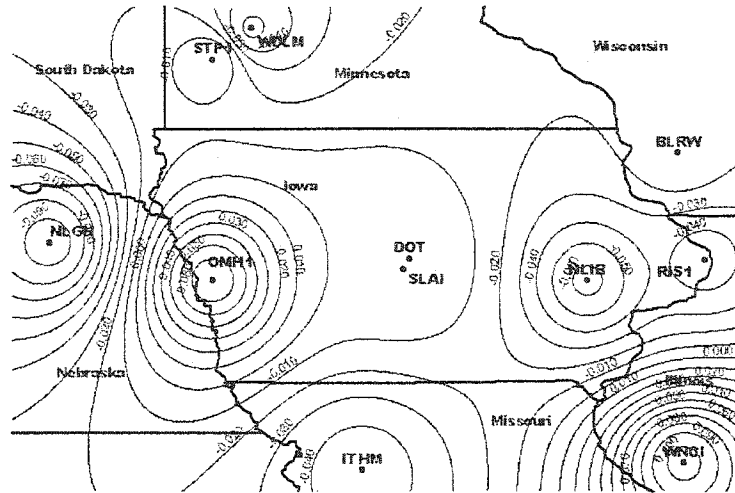


Fig. 5.13 Difference between Day282 and Day85

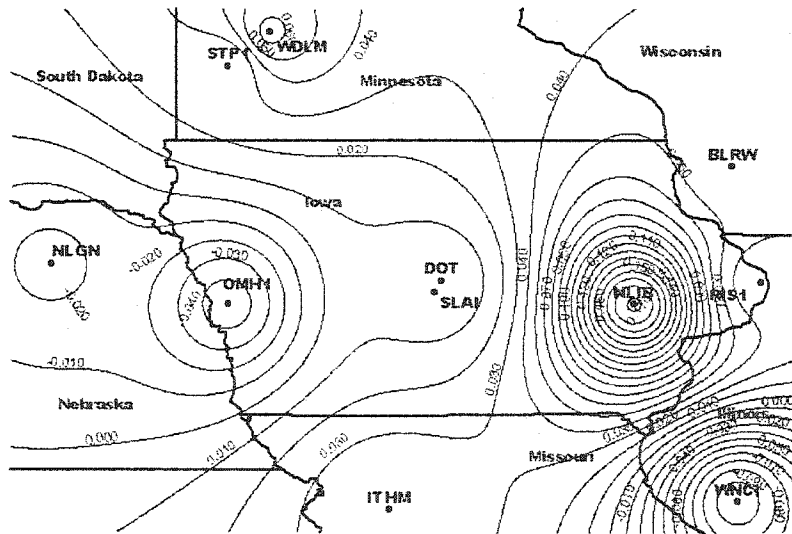


Fig. 5.14 Difference between Day78 and Day282

Fig. 5.12 to Fig. 5.14 shows that the geoid changes between two observation times. The changes are significant. We also can see the GEOID changes in direction and amount both. Those changes are different at each point at different time period. It shows that GEOID model is time related model. One can't use simple linear interpolation or normal polynomial model to describe it. All these changes are also not directly relative to the height of each point, which are not a simple linear function of heights. The lower height points may have higher GEOID change rate, such as point NLIB. Obviously the GEOID96 or GEOID99 model is not good enough to meet our engineering application accuracy, and the point density of CORS does not satisfy the accurate geoid model too.

CHAPTER 6. MODELING LOCAL GEOID WITH GPS AND LEVELING

6.1 Introduction

Due to the wide use of the GPS satellite navigation system in surveying and geodesy, the transformation between the geometrical or ellipsoidal heights (obtained by GPS) and the orthometric heights or normal heights (obtained by geodetic leveling) has become increasingly important.

The difference between the ellipsoidal height and the orthometric height corresponds to the geoid undulation. For high precision geodetic applications, this undulation must be determined with an accuracy of a few cm to 10cm over a distance of 100 to 200 km, and for engineering applications the accuracy should reach 1cm to 3cm over 10km-20km. This research presents the basic investigations for the geoid

determination in Story County, Iowa and the practical computations to reach this goal.

Different methods for determining the geoid from gravity measurements or from astrogeodetic observations have existed since the last century. The new high accuracy geoid local model obtained in this research is combined with modern satellite observations, geodetic leveling, and national GEOID96, GEOID99 models.

The geoid model is computed in three parts using a remove-restore technique (Rapp and Rummel, 1975).

$$N = N_1 + N_2 + N_3 + \varepsilon_N$$

$$N = \frac{R}{4\pi\gamma} \iint_{\sigma} \Delta g_{gm} S(\phi) d\sigma + \frac{R}{4\pi\gamma} \iint_{\sigma_0} (\Delta g - \Delta g_{gm}) S(\phi) d\sigma + N_3 + \varepsilon_N \quad 6.1$$

The first term is the contribution to the gravity anomalies from a global geopotential model (Δg_{gm}), which is about 1-0.5m accuracy, the second term is the contribution from the surface gravity measurements (Δg), less the geopotential contribution for the computational area σ_0 , which is about 0.1 to 0.3m accuracy, N_2 is the primary indirect effect on the geoid produced by the condensation of the masses above the

geoid, and ε_n is the truncation error, i.e. the geoid height information missing from gravity data outside the computational region ($\sigma-\sigma_0$). The truncation error is neglected for the computation, but it can be significant.

The geoid model cannot be computed directly because the integral terms N_1 and N_2 cannot be evaluated analytically. Therefore, a series of assumptions and approximations are applied to equation (6.1) in order to determine the geoid model. The theory assumes that the reference ellipsoid has the same potential as the geoid and the same mass as the earth. The geoid model will be biased by a constant N_0 . In addition, it is assumed that the center of the ellipsoid coincides with the center of gravity of the earth. Thus, the Geoid96 or Geoid99 will not be compatible with the North American Datum of 1983 (NAD83).

The term N_1 is computed from a spherical harmonic expansion as follows (Gunter Seeber 1993):

$$N_1 = N_{gm} = \frac{GM}{r\gamma} \sum_{n=2}^{360} \sum_{m=0}^n \left(\frac{a}{r}\right)^n (C_{nm} \cos m\lambda + S_{nm} \sin m\lambda) P_{nm}(\sin \phi) \quad 6.2$$

where γ is the normal gravity on the reference ellipsoid at the geodetic latitude of the computation point, r is the geocentric radial

distance of the computation point projected on the ellipsoid, a is the semi-major axis, ϕ is the latitude and λ is the longitude. The fully normalized harmonic coefficients, C_{nm} and S_{nm} , are obtained from the geopotential model to degree and order 360 (Rapp et al., 1991). Finally, P_{nm} are the fully normalized associated Legendre polynomials.

The determination of N_2 is the most complex task in equation (6.1). It involves two important approximations:

- the gravity reductions are applied in a planar approximation
- the gravity anomalies are linearly correlated to the topography
- the gravity anomalies Δg_{gm} are determined from a spherical harmonic expansion:

$$\Delta g_{gm} = \frac{GM}{r^2} \sum_{n=2}^{360} \sum_{m=0}^n (n-1) \left(\frac{a}{r}\right)^n (C_{nm} \cos m\lambda + S_{nm} \sin m\lambda) P_{nm}(\sin \phi) \quad 6.3$$

We have the following expression of total Geoid height between GPS ellipsoidal height and orthometric height:

$$h = H + N1, 2(\text{Geoid96 or GEOID99}) + N3(\text{local}) \quad 6.4$$

where h is ellipsoidal heights from GPS, and H is orthometric height from geodetic leveling.

It is reasonable to interpret the geoid correction from the difference between GPS and leveling heights after adding GEOID96 or Geoid99. As described above, these discrepancies will contain error contributions from GPS and leveling, as well as from the geoid model itself.

The local geoid model accuracy we expect is in the range 1–3 cm ± 1 sigma. It will be satisfactory for most geodetic surveying and engineering applications, such as a highway project. The geodetic leveling accuracy is:

	<u>First order</u>	<u>Second order</u>	<u>Third order</u>
Accuracy	$5.0\text{mm}\cdot\sqrt{L}$	$8.0\text{mm}\cdot\sqrt{L}$	$12.0\text{mm}\cdot\sqrt{L}$

Where L = the distance in unit of km between two points. If the distance between two points are about the 5km to 10km, so the accuracy expectation are better than a third and close to a second order level.

6.2. Principle of the Kalman Filter

Fig. 6.1 is a typical GPS position plan. When we establish a new GPS project, the plan is:

- set new GPS points along the project area. Distances between points depends on project, e.g. if the distance is 1 to 5 miles, then we collect data 30 to 90 minutes
- connect these GPS points to HARN reference points, which are within 10 to 50 miles
- connect these GPS points to CORS as reference points, which are within 200 to 400 miles

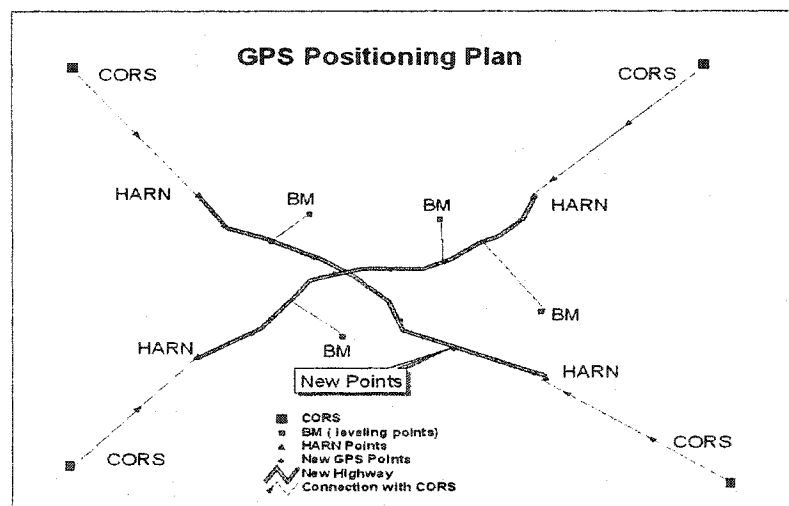


Fig. 6.1 A Typical GPS Positioning Plan

- connect these GPS points to some BM (leveling points) for height reference. Those leveling points are established in different years, different order (accuracy), and sometimes different datum

In this research the Kalman Filter has been used to fit the local geoid model. The main reasons for this choice are:

- the geoid model is relative to earth gravity, where as the neighborhood points affect each other
- we not only need to fit geoid correction for existing points, but we also need to predict geoid correction for new points.
- when we get new observations, we need to update our model and adjust the predicted values for new points
- both observation and state parameters have random sequences

Because of reiteration observations in the reference station (HARN or CORS), geoid model is a dynamic system. This means that Geoid undulations will change even without observation errors.

Suppose the Kalman filtering State equation and Observation equation are [156]

$$\underset{n \times 1}{X_k} = \underset{n \times n}{\Phi_{k,k-1}} \underset{n \times 1}{X_{k-1}} + \underset{n \times r}{\Gamma_{k-1}} \underset{r \times 1}{\Omega_{k-1}} \quad 6.5$$

$$\underset{m \times 1}{L_k} = \underset{m \times n}{A_k} \underset{n \times 1}{X_k} + \underset{m \times 1}{\varepsilon_k} \quad (k = 1, 2, \dots)$$

where X_k is n-by-1 dimension state vector, $\Phi_{k,k-1}$ is n-by-n state transition matrix. Ω_{k-1} is dynamic noise vector. L_k is m-by-1 observation vector. A_k is m-by-n observation matrix. ε_k is observation noise vector. (Ω_{k-1}) and (ε_k) are assumed to be mutually uncorrelated and uncorrelated between sampling times. Γ_{k-1} is a n-by-r dynamic noise transition matrix.

Kalman basic equation:

The state one-step prediction equation is:

$$\hat{X}_{K/K-1} = \Phi_{K,K-1} \hat{X}_{K-1} \quad 6.6$$

The state estimate equation is:

$$\hat{X}_K = \hat{X}_{K/K-1} + K_K (L_K - A_K \hat{X}_{K/K-1}) \quad 6.7$$

The optimal filter gain matrix is

$$K_K = D_{K/K-1} A_K^T (A_K D_{K/K-1} A_K^T + R_K)^{-1} \quad 6.8$$

The one-step prediction mean-square covariance matrix is:

$$D_{K/K-1} = \Phi_{K,K-1} D_{K-1}^T \Phi_{K,K-1}^T + \Gamma_{K-1} Q_{K-1} \Gamma_{K-1}^T \quad 6.9$$

The covariance matrix is

$$D_K = (1 - K_K A_K) D_{K/K-1} (1 - K_K A_K)^T + K_K R_K K_K^T \quad 6.10$$

The observations:

h - heights from GPS, ($i = 1, \dots, m$), which include observation errors, errors from reference points, and from data processing

H - heights from geodetic leveling, ($i = 1, \dots, m$), which include observation errors, but we cannot reprocess the data, bias from datum.

The observation equation are

$$h-H - \text{Geoid99} = f(\text{lat.}, \text{long.}, \text{Geoid99}) + \varepsilon$$

$$h-H - \text{Geoid99} = AX + \varepsilon \quad 6.11$$

The State parameters are:

N_3 (local geoid correction model vector (m-by-1)), which can be determined by a different fitted geoid undulation model. From above eq. 6.5, we know the following relationship for common points,

$$N_{\text{new}} = N_{\text{old}} + \text{error}(\text{geoid99}, \text{model_errors})$$

$$\text{error}(\text{geoid96}, m_error) = \frac{1}{R} \frac{\partial N}{\partial \text{lat.}} + \frac{1}{R \cos(\text{lat.})} \frac{\partial N}{\partial \text{long.}} \quad 6.12$$

which is the components of geoidal deflection of vertical at a point (Lat. Long.).

The problems are:

- this is a nonlinear function
- how to define the state equation, which gives the relationship between state parameters.
- how to define dynamic noise
- how to define covariance matrix of observation and dynamic noise
- how to chose initial vector of state
- how to determine variance and covariance matrix of state

Interpolation and smooth problem

The limitations of interpolation and smooth are:

- the accuracy of interpolation and smooth
- the method of GPS data processing
- the error factors in GPS height

The method of model update

With time, more and more points will have reiteration data, because of observation errors, geoid change, and the data processing method, the heights will be different between them. The problem is how to reconcile those differences.

6.3 Modeling of Local Geoid by Polynomial**Surface Fitting**

Parameters describing the earth's gravity field are most often used to determine the general shape of the geoid over a large area. The model for the large area will be more complex and less accurate.

However for the purpose of geodetic control in a small area covered by an engineering survey requiring high precision, local variations in the earth's gravity field must be considered. Some approaches, as described in section 6.1, may not be able to provide the required

accuracy. With a large number of monuments, where both the GPS ellipsoidal heights and the orthometric heights from geodetic leveling have been observed, the geoidal heights at these points can be approximated by using the following simple relation

$$N = h - H \quad 6.13$$

where:

N is the geoidal height or undulation

h is the ellipsoidal height from GPS surveys, and

H is the orthometric height from geodetic leveling.

Note that in Equation (6.13) the curvature of both the plumb line and the ellipsoidal normal is neglected (Fig. 6.2)[165]. Although the

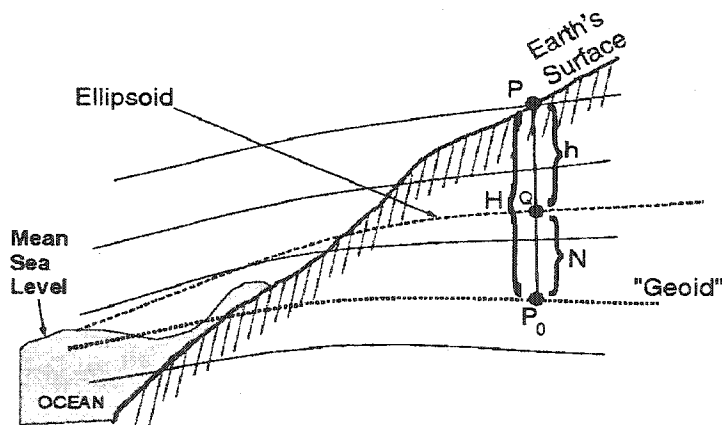


Fig. 6.2 The Relationship between GPS Height and Orthometric Height

formula is approximate, the error is negligible for practical applications. Comprehensive studies of geoid determination with GPS and leveling are found in Zilkoski (1993; 1990a; 1990b) and Zilkoski and Hothem (1988), amongst others.

Modeling geoid has two main purposes, first, one is to correct GPS height to geodetic height for existing points, and second and more important, to predict and interpolate geoid correction for new points by the model. In order to predict and interpolate the geoidal heights at new points where the actual surveying data are not available, one may approximate the geoid by a best fitting surface, for instance, and express the geoidal height, $N(x, y)$, at a point (x, y) by a polynomial of order n

$$N = f(x, y) = \sum_{i,j} a_i x^i y^{n-j} \quad 6.14$$

where (x, y) are the horizontal coordinates of a point in a local coordinate system. n is the order of polynomial.

$$i = 0, 1, \dots, ((n+1)^2 + (n+1))/2, \quad j = 0, 1, 2, \dots, n \quad 6.15$$

The observed values, which are functions of position, are used to determine the coefficients of the best fitting polynomial through application of the least squares approximation technique as discussed below.

6.4 Basic Principles of the Least Squares Trend

Analysis

To implement the polynomial surface fitting technique for modeling a local geoid as described above, the basic principles of least squares trend analysis are reviewed here.

Estimation of Unknown Parameters

Consider the following general nonlinear observational model:

$$f(x, l) = 0 \quad 6.16$$

where

f is an m by 1 vector of nonlinear functions

x is a u by 1 vector of unknown parameters to be solved, and

l is an n by 1 vector of true values of the observations.

Linearizing Equation (6.16), one obtains:

$$AV + B\Delta = f \quad 6.16$$

for the nonlinear case

$$A = \frac{\partial F}{\partial l} \quad B = \frac{\partial F}{\partial X} \quad f = -F(L, X_0)$$

X_0 is the vector of approximate values of x

L is the vector of actual observations, and

The least squares solution is

$$\begin{aligned} N &= B^T(AQA^T)^{-1}B \\ U &= B^T(AQA^T)^{-1}f \\ \Delta &= N^{-1}U \\ X &= X_0 + \Delta \end{aligned} \tag{6.17}$$

$$\begin{aligned} V &= QA^T(AQA^T)^{-1}(f - B\Delta) \\ \hat{l} &= l + V \end{aligned} \tag{6.18}$$

the cofactor matrices are

$$\begin{aligned} Q_{\Delta\Delta} &= Q_{XX} = N^{-1} \\ Q_{VV} &= QA^T(AQA^T)^{-1}(I - BN^{-1}B^T(AQA^T)^{-1})AQ \\ Q_{\hat{l}\hat{l}} &= Q - Q_{VV} \end{aligned} \tag{6.19}$$

the estimation of the reference variance is

$$\hat{\sigma}_o^2 = \frac{V^T PV}{m - u} \tag{6.20}$$

where Q is the given variance-covariance matrix of the observations.

6.5 Statistical Analysis of the Results

To analyze the local geoid model, we use some statistical techniques. The “goodness” of fit is a basic statistical test. This test the

significance of the estimated parameters, and the residuals to find out the outliers.

Goodness of Fit Test

The appropriateness of an estimated trend function can be tested by using the following statistic:

$$\frac{(m-u)\hat{\sigma}_o^2}{\sigma_o^2} \sim \chi^2(m-u) \quad 6.21$$

where

σ_o^2 is a posterior variance factor from the adjustment of the observations.

m and u are same as above, which means the total numbers of independent equations and unknown parameters.

$\hat{\sigma}_o^2$ is the estimated variance factor of the fitting given by

Eq.6.20

and $\chi^2(m-u)$ is the chi-square function

By using the significance level α , we could test the selected trend function and determine which fit is appropriate if the following inequality holds:

$$\frac{(m-u)\hat{\sigma}_0^2}{\sigma_0^2} \leq \chi^2(m-u) \quad 6.22$$

Significance Testing of Estimated Parameters

Significance testing of each of the estimated parameters in the trend function can be performed using the following statistic:

$$\frac{\hat{x}_i - \mu_{\hat{x}_i}}{\sigma_{\hat{x}_i}} \sim n(0,1) \quad 6.23$$

where

$\mu_{\hat{x}_i}$ is the expected value of \hat{x}_i

$\sigma_{\hat{x}_i}$ is the standard deviation of the estimated parameter \hat{x}_i

$n(0,1)$ is the standard normal distribution function.

An estimated parameter is considered as being significant if for a selected significance level α

$$\left| \frac{\hat{x}_i}{\sigma_{\hat{x}_i}} \right| \geq n_{1-\alpha/2}(0,1) \quad 6.24$$

Any non-significant parameters are deleted from the initially selected trend function.

Test of residuals

Individual observations which do not follow the global trend as defined by the majority of the data can be tested by the following statistic:

$$\frac{\hat{v}_i - \mu_{\hat{v}_i}}{\sigma_{\hat{v}_i}} \sim n(0,1) \quad 6.25$$

where $\mu_{\hat{v}_i}$ is the expected value of \hat{v}_i , and $\sigma_{\hat{v}_i}$ is the standard deviation of the estimated residuals.

Using the significance level α , we could find any outliers which do not follow the global trend by above test.

6.6 Local Geoid Model for Story County

To study the performance of the technique of polynomial surface-fitting and Kalman local geoid modeling in Story County of Iowa, four sets of data have been collected for 11 points which have geodetic or orthometric height by leveling. By GPS processing, we have latitude, longitude and GPS height for each point. All points cover an area of

approximately 20x20 miles. The relative locations of the network stations are shown in Figure 6.3.

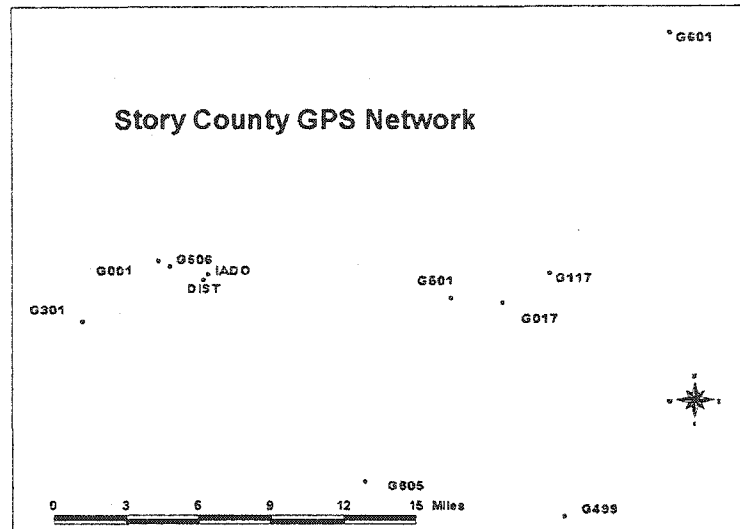


Figure 6.3. The Geodetic Control Network.

To develop the Story County geoid model to meet engineering application standard, we and the Iowa DOT have collected GPS data at 11 points around Story County on day 258 of 2000, day85 and day86 of 2001, day282 of 2001, day78 of 2002. The observations were in two sessions, which could be on the same day or in two days. If it was on one day, then the observations were made in the morning and afternoon. In each session observation were done in three to four hours at the same time. After observation, all data was processed by DGPS with precise ephemeris. During the processing, all the cycle slips were

fixed. The processing method is the carrier phase IONO free combination. Each processing chose IDOT, G117 and G506 as fixed reference point. After obtaining all vector and coordinate solutions, we converted results of G117 and G506 as reference points to IDOT as reference point data set. After this conversion, all three results were averaged to get a final result with IDOT as a reference point [table 6.1]. All the geoid corrections were computed using both GEOID96 and GEOID99 model.

From Table 6.1, the mean of all geoid96 corrections is -29.8245m. The mean of all geoid99 corrections is -29.8584m. The

Table 6.1 Story County Geoid Network Results

Station	E.ht.258	E.Ht.85	E.Ht.282	E.Ht.78	H.Elev	Geoid96	Geoid99
IADOT	263.7419	263.7419	263.7419	263.7419	292.7490	-29.3050	-29.3490
*DIST	244.1119	244.1119	244.0948	244.0899	273.1197	-29.3190	-29.3620
G601	299.8664	299.8674	299.8760	299.8803	329.5630	-29.9650	-30.0000
G605	234.0619	234.0822	234.0884	234.0828	264.3690	-30.5420	-30.5660
G301	285.5212	285.5422	285.5174	285.5090	314.3160	-29.1400	-29.1820
G117	277.5993	277.6033	277.6199	277.5946	307.5250	-30.2720	-30.2990
G501	273.8959	273.8921	273.8761	273.8970	303.5840	-30.1060	-30.1330
G001	264.4660	264.5410	264.5854	264.5835	293.2240	-29.0790	-29.1190
G506	258.2811	258.2739	258.2979	258.2881	287.1130	-29.1410	-29.1820
G499	232.2807	232.2710	232.2709	232.2789	262.8940	-30.9490	-30.9730
G017	269.8957	269.9261	269.9857	269.9934	299.8107	-30.2510	-30.2770

differences between two geoid model show as -0.044, -0.043, -0.035, -0.024, -0.042, -0.027, -0.027, -0.040, -0.041, -0.024, -0.026.

The mean of all these differences is -0.034m. It is significant that there is a bias between GEOID996 and GEOID99 or the GEOID may have changed in this area during these years.

Fig. 6.4 is the GEOID99 model for the Story County area. From the GEOID99 of Story County, we can see that there is a slope from south-east to north-west. The difference between highest east end point and lowest west end point is about one and half meters. This is not relative to the height value of each point.

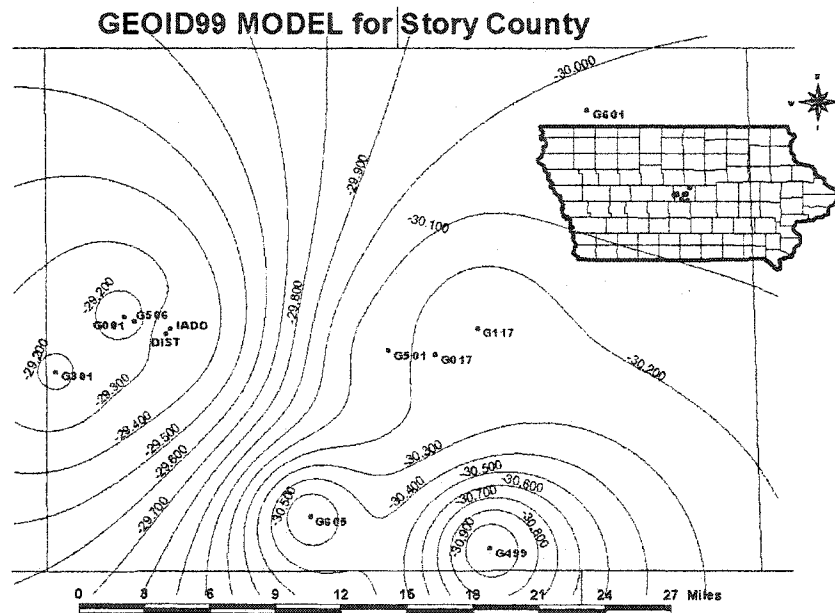


Fig. 6.4 GEOID99 Model for Story County Area.

Using the GEOID99 model, we can compute GPS orthometric heights for all 11 points [table 6.2]. We call this height, Height99. We can also use GEOID96 model to compute orthometric heights for all 11 points. We call this height, Height96. The difference between Height99 and Height96 actually is the difference between model GEOID99 and GEOID96. Fig. 6.5 shows the differences between GEOID99 and GEOID96.

Table 6.2 The Orthometric Heights from GEIOD99

Station	Elev Height	Day258 Height99	Day85 Height99	Day282 Height99	Day78 Height99
IADOT	292.7490	293.0909	293.0909	293.0909	293.0909
*DIST	273.1197	273.4739	273.4739	273.4568	273.4519
G601	329.5630	329.8664	329.8674	329.8760	329.8803
G605	264.3690	264.6279	264.6482	264.6544	264.6488
G301	314.3160	314.7032	314.7242	314.6994	314.6910
G117	307.5250	307.8983	307.9023	307.9189	307.8936
G501	303.5840	304.0289	304.0251	304.0091	304.0300
G001	293.2240	293.5850	293.6600	293.7044	293.7025
G506	287.1130	287.4631	287.4559	287.4799	287.4701
G499	262.8940	263.2537	263.2440	263.2439	263.2519
G017	299.8107	300.1727	300.2331	300.2627	300.2704

By Comparing the Fig.6.4 and Fig.6.5 we find out that they both have slope, but the direction is different. The Fig.6.5 shows that the difference between GEOID99 and GEOID96 in the west end is bigger than the difference in the east end. There is also a bias of about 3 cm between the two models. This means the geoid changes from time to time.

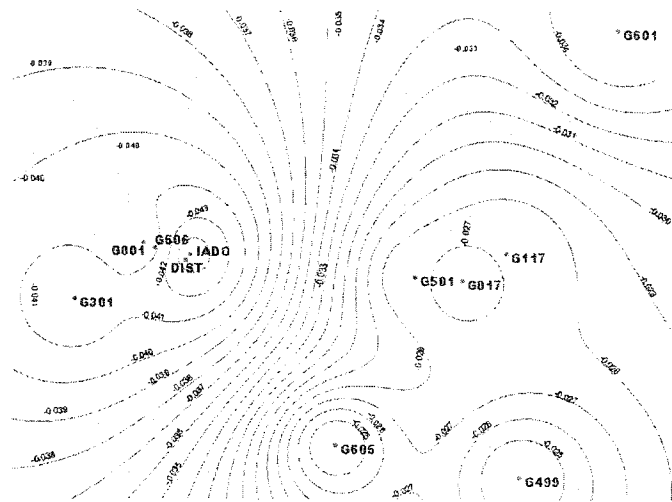


Fig. 6.5 The Difference between GEOID99 and GEOID96

The differences between true orthometric heights and converted orthometric heights from GPS heights are listed in table 6.3.

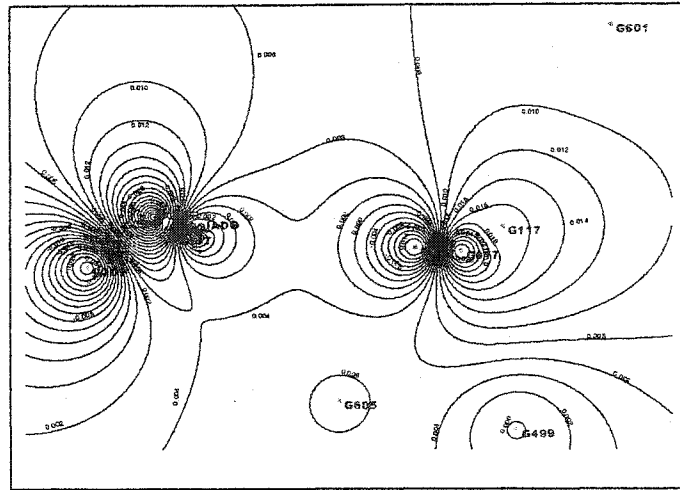


Fig. 6.6 The Difference between Day85 and Day285

Table 6.3 The Difference between GPS Converted Orthometric Heights
and Geodetic Leveling Heights

Station	Elev-GPS Diff.	2001-2000 E85-E285	2001-2001 E282-E85	2002-2001 E78-E282
IADOT	-0.3419	0.0000	0.0000	0.0000
DIST	-0.3542	0.0000	-0.0171	-0.0049
G601	-0.3034	0.0010	0.0086	0.0043
G605	-0.2589	0.0203	0.0062	-0.0056
G301	-0.3872	0.0210	-0.0248	-0.0084
G117	-0.3733	0.0040	0.0166	-0.0253
G501	-0.4449	-0.0038	-0.0160	0.0209
G001	-0.3610	-0.0750	0.0444	-0.0019
G506	-0.3501	-0.0072	0.0240	-0.0098
G499	-0.3597	-0.0097	-0.0001	0.0080
G017	-0.3620	0.0604	0.0296	0.0077
Mean	-0.3542	0.0107	0.0104	-0.0014

After we obtain all orthometric heights we can compute the changes among different observation time. Fig. 6.6 to Fig.6.9 show the changes among different observation time.

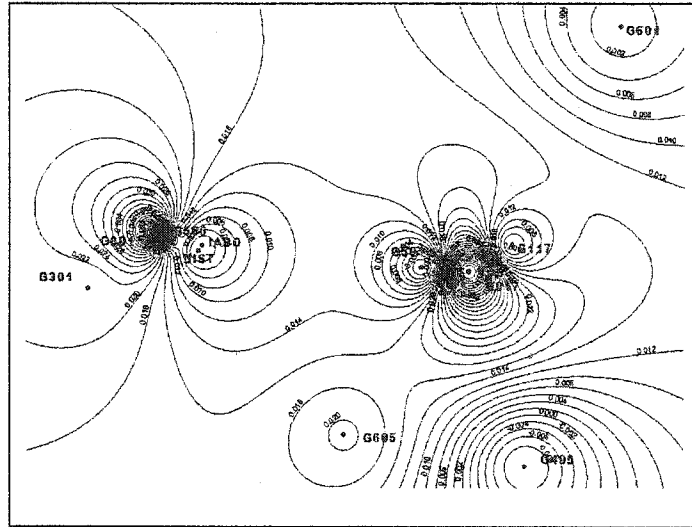


Fig. 6.7 The Difference between Day282 and Day85

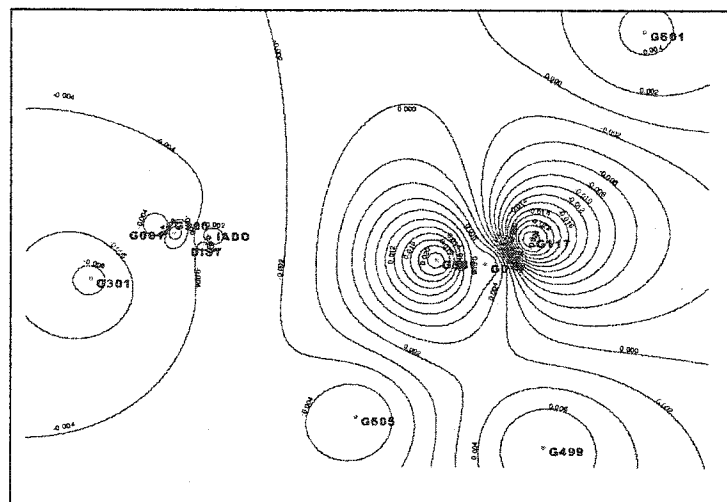


Fig. 6.8 The Difference between Day78 and Day282

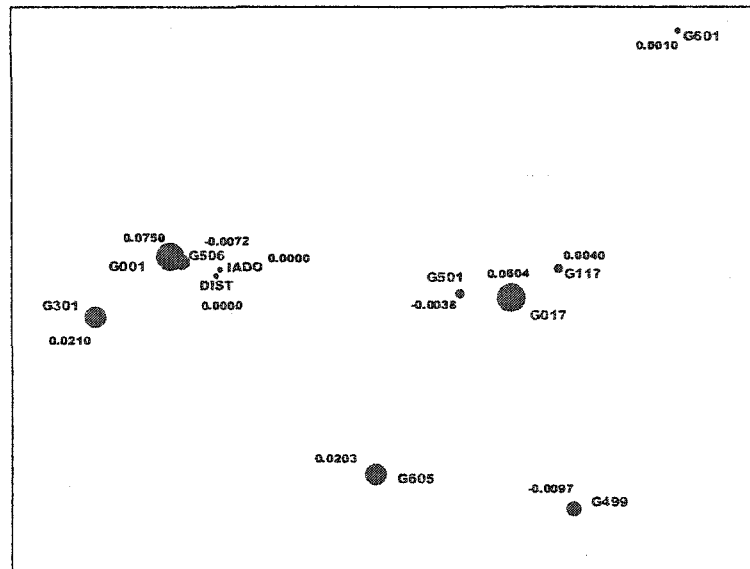


Fig. 6.9 The Difference between Day85 and Day285

Fig. 6.9 shows the difference between Day85 and Day285 by the size of dots.

Selection of a Trend Function for the Local Geoid

Assuming that geoid surface is quite flat in the relatively small area of the project, a third order polynomial is chosen to analyze the trend of the geoidal undulations for this area. It is expressed as follows:

$$H_{GPS_k} - N_{99_k} - h_{leveling_k} = \sum_{i,j} a_i x_k^i y_k^{n-j} \quad 6.26$$

where (x_k, y_k) are the horizontal coordinates of the k th point,

N_k is the GEOID99 correction at the k th point, and

a_i are the unknown parameters to be estimated.

Considering each (x_k, y_k, N_k, h_k) ($k=1, 2, \dots$) data set as an observed data point, we obtain unknown parameters corresponding to above model.

We could also choose Kalman filter method for the local geoid model as we discussed above.

$$H_{GPS_k} - N_{99_k} - h_{leveling_k} = \sum_{i,j} a_i x_k^j y_k^{n-j} + V_m + \sum V_{g_l} \quad 6.26$$

where (x_k, y_k) are the horizontal coordinates of the k th point,

N_k is the GEOID99 correction at the k th point,

a_i are the unknown parameters to be estimated,

V_m is the mean velocity of all points in the project area,

V_{g_l} is the mean velocity of group L points in the project area.

The number of groups in the estimated area depends on the project area. In this research, we divided all 11 points into three groups, G001 as group one, G017 as group two and remaining points in the third group.

Considering each (x_k, y_k, N_k, h_k) ($k=1,2, \dots$) data set as an observed data point, one obtains the coefficient matrices and vectors corresponding to above model.

6.7 Estimation of Unknown polynomial Coefficients

As we described above, for all 11 points both GPS ellipsoidal heights and leveled (orthometric) heights are available. These points are denoted in Figure 6.3. In Figure 6.3 locations of these common points have been carefully designed to ensure that they are evenly distributed through the project area and meet engineering application requests. The GPS survey was done according to the accuracy standard of the Federal Geodetic Control Subcommittee (FGCS). The data collection procedures were described earlier. There are four data sets, and each data set includes two sessions, data collected in one day or two days. The geodetic leveling is according to the FGCS too, but is from a different order and year. Accuracy analysis of these survey campaigns shows that the relative GPS heights obtained is of an accuracy better

than 1cm level, and the accuracy of the obtained relative orthometric heights obtained by geodetic leveling is better than third order.

Using the trend analysis and Kalman techniques described above, the origin of the local x, y (longitude, latitude) coordinate system is chosen at approximately the center of the project area, i.e., the average of all points, and the new xy-coordinates are calculated according to following equations:

$$\begin{aligned} x_i &= R \cdot (lon_i - lon_o) \cdot \cos(lat_o) \\ y_i &= R \cdot (lat_i - lat_o) \end{aligned} \quad 6.27$$

where R is the mean radius of curvature of the earth, (lat_o, lon_o) are the latitude and longitude of the origin, and x_i, y_i are local coordinates of the point i.

As is well known, GPS gives precise ellipsoidal height differences instead of ellipsoidal heights. Therefore, effort is made here to model the geoidal height differences instead of geoidal heights. This is easily done by replacing N_i by dN . The origin is then excluded during estimation processing. According to the equations developed above, and the accuracy of the GPS and orthometric height differences, the estimated coefficients of the polynomial (Eq. 6.26) are listed in Tables

6.4, 6.5, and 6.7, from which we can see that all the estimated parameters are significant. The estimated posterior variance factor is 0.018m to 0.050m. The estimated posterior variance factors for each set itself are 0.0018, 0.0165, 0.0169, and 0.0186 meters. This means that there is a good geoid surface fit for each data set at observed time if we only look at the estimated posterior variance factor. But if we check the residuals for each point, we find out that the residuals at G001 and G506 in day85, day282, and day78 are bigger than 3cm and significantly bigger than for the rest of points. This means that the polynomial model is not satisfactory for engineering application accuracy in the Story County area which needs an accuracy of 1cm to 3cm and better than third order leveling.

We find that the polynomial does not work well for prediction; the estimated posterior variance factor varies from 0.018 to 0.050 meter (show as "dayXXX from dayXXX" in table 6.4 to table 6.7), far bigger than the variance factor using the observation time model, which passed the goodness of fit test at 95% confidence level. The maximum allowable value is 0.038m. In addition, except for two residuals, point G001 and G506, all the estimated residuals passed the statistical testing. The estimated linear function is therefore

statistically adequate to represent the shape of the geoid in the project area.

The geoid models for each observation day are given in table 6.4 through 6.7. In the table, "W" mean weight, "V" means residuals for each point, the unit is in meter. "V85 from 282" means the residuals using model of day282 to interpolate day85. Other tables are similar.

Table 6.4 The Estimated Polynomial Coefficients for Day258, 2000

 Polynomial coefficients a0 to a9:

0.438, -0.974, -10.215, 116.897, 0.789, -23.005, 410.049, -92.960, -667.294, -762.786

IADOT *DIST G601 G605 G301 G117 G501 G001 G506 G499 G017

W: 1.0000 1.0000 1.0000 1.0000 1.0000 1.0000 1.0000 1.0000 1.0000 1.0000

V: -0.0000 -0.0012 0.0000 0.0000 -0.0000 -0.0001 0.0001 -0.0031 0.0046 -0.0000 0.0001

Std.258 = 0.0018 m

Table 6.5 The Estimated Polynomial Coefficients for Day85, 2001

Polynomial coefficients ao to a9:

0.398, 1.267, -3.262, 92.084, -0.638, 10.125, 227.249, -35.680, -228.956, -162.204

	IADOT	*DIST	G601	G605	G301	G117	G501	G001	G506	G499	G017	
W	:	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	
V85	:	-0.0000	-0.0108	0.0000	0.0001	-0.0002	-0.0005	0.0006	-0.0286	0.0421	-0.0001	0.0005
V85 from 258:		-0.0000	-0.0012	-0.0010	-0.0203	-0.0210	-0.0041	0.0039	-0.0781	0.0118	0.0097	-0.0603

Std.85 = 0.0165 m Std.85_from_258 = 0.0330 m

Table 6.6 The Estimated Polynomial Coefficients for Day282, 2001

Polynomial coefficients ao to a9:

0.345, -1.604, 1.822, 72.891, 0.766, 34.187, -3.114, 32.736, 218.276, 295.099

	IADOT	*DIST	G601	G605	G301	G117	G501	G001	G506	G499	G017	
W	:	1.0000	0.9818	0.9954	0.9976	0.9788	0.9829	0.7225	0.0996	0.2978	1.0000	0.1043
V282	:	-0.0000	-0.0111	0.0000	0.0001	-0.0002	-0.0005	0.0007	-0.0293	0.0433	-0.0001	0.0005
V282 from 85:		-0.0000	0.0063	-0.0086	-0.0061	0.0246	-0.0171	0.0166	-0.0730	0.0183	0.0000	-0.0291
V282 from 258:		-0.0000	0.0159	-0.0096	-0.0265	0.0038	-0.0207	0.0199	-0.1225	-0.0122	0.0098	-0.0899

Std.282 = 0.0169 m Std.282_from_85 = 0.0280 m Std.282_from_258 = 0.0502 m

Table 6.7 The Estimated Polynomial Coefficients for Day78, 2002

Polynomial coefficients a_0 to a_9 :

0.362, -1.866, -1.001, 90.220, 1.666, 38.137, -95.057, 52.468, 295.460, 418.276

	IADOT	*DIST	G601	G605	G301	G117	G501	G001	G506	G499	G017
W	: 1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
V78	: -0.0000	-0.0122	0.0000	0.0002	-0.0002	-0.0006	0.0007	-0.0322	0.0476	-0.0001	0.0005
V78 from 282:	-0.0000	-0.0062	-0.0043	0.0057	0.0082	0.0248	-0.0202	-0.0274	0.0531	-0.0081	-0.0072
V78 from 85:	-0.0000	0.0112	-0.0129	-0.0005	0.0330	0.0082	-0.0043	-0.0711	0.0281	-0.0080	-0.0368
V78 from 258:	-0.0000	0.0208	-0.0139	-0.0209	0.0122	0.0046	-0.0010	-0.1206	-0.0024	0.0018	-0.0976
Std.78 = 0.0186 m	Std.78_from_282 = 0.0221 m										
Std.78_from_85 = 0.0295 m	Std.78_from_258 = 0.0503 m										

For the Kalman filter model, from equation 6.5 to 6.11, we can have kalman model as,

State transfer equation

$$X_K = \Phi_{13 \times 13} X_{K-1} \quad \Phi_{13 \times 13} = I_{13 \times 13}$$

Observation equation

$$L_k = A_k X_k + \varepsilon_k \quad (k = 1, 2, \dots)$$

$\begin{matrix} 11 \times 1 & 11 \times 14 & 14 \times 1 & 14 \times 1 \end{matrix}$

$$L_K = [l_1 \quad \dots \quad l_n]_{11 \times 1}^T \quad i = \text{day258} \quad \text{day85} \quad \text{day282} \quad \text{day78}$$

For the dynamic model, we could have

$$Q_i = \begin{bmatrix} 0.001 & \dots & 0 \\ \vdots & \ddots & \vdots \\ 0 & \dots & 0.001 \end{bmatrix}_{11 \times 11} \quad i = \text{day258} \quad \text{day85} \quad \text{day282} \quad \text{day78}$$

For the observation noise model, we have

$$R_i = \begin{bmatrix} r_{1,1} & \dots & r_{1,12} \\ \vdots & \ddots & \vdots \\ r_{12,1} & \dots & r_{12,12} \end{bmatrix}_{11 \times 11} \quad i = \text{day258} \quad \text{day85} \quad \text{day282} \quad \text{day78}$$

which can be obtained from polynomial model estimation.

For the initial variance and covariance matrix of state X, we can get it from polynomial model, or we can set it as our required accuracy.

$$D_i = \begin{bmatrix} 0.001 & \dots & 0 \\ \vdots & \ddots & \vdots \\ 0 & \dots & 0.001 \end{bmatrix}_{11 \times 11} \quad i = \text{day258} \quad \text{day85} \quad \text{day282} \quad \text{day78}$$

Initial state X can be obtained from first observation and second observation vector.

$$X_0 = \begin{bmatrix} X_{\text{day258}} \\ X_V \end{bmatrix}_{13 \times 1}$$

there

$$X_{\text{day258}} = [a_0 \quad a_1 \quad a_2 \quad a_3 \quad a_4 \quad a_5 \quad a_6 \quad a_7 \quad a_8 \quad a_9]^T$$

$$X_V = [v_{\text{mean}} \quad v_8 \quad v_{11}]^T$$

v_{mean} --- mean velocity of all points, which also is the local model basic velocity.

v_8 --- the velocity of point group1 (G001)

v_{11} --- the velocity of point group2 (G117)

$$A_t = \begin{bmatrix} A_t^0 & A_v \end{bmatrix}_{11 \times 14}$$

There A_t^0 is static observation matrix from previous polynomial model.

It describes a static local geoid model. We can get it from our polynomial model from day258. The second part is kinematic part of our local geoid model, which describe the changes of our local geoid model. In the small area, we can set whole area as a block, or we can set whole area as a few groups. In our case, after we analyzed the data and results from polynomial model we split the area as three groups.

$$A_k = \begin{array}{cccccccccccc} 1.0000 & 0.0885 & 0.0078 & 0.0007 & 0.0159 & 0.0014 & 0.0001 & 0.0003 & 0.0000 & 0.0000 & dt & 0 & 0 \\ 1.0000 & 0.0913 & 0.0083 & 0.0008 & 0.0125 & 0.0011 & 0.0001 & 0.0002 & 0.0000 & 0.0000 & dt & 0 & 0 \\ 1.0000 & -0.1870 & 0.0350 & -0.0065 & 0.1598 & -0.0299 & 0.0056 & 0.0255 & -0.0048 & 0.0041 & dt & 0 & 0 \\ 1.0000 & -0.0049 & 0.0000 & -0.0000 & -0.1080 & 0.0005 & -0.0000 & 0.0117 & -0.0001 & -0.0013 & dt & 0 & 0 \\ 1.0000 & 0.1639 & 0.0269 & 0.0044 & -0.0125 & -0.0021 & -0.0003 & 0.0002 & 0.0000 & -0.0000 & 0 & 0 & 0 \\ 1.0000 & -0.1151 & 0.0132 & -0.0015 & 0.0163 & -0.0019 & 0.0002 & 0.0003 & -0.0000 & 0.0000 & 0 & 0 & 0 \\ 1.0000 & -0.0562 & 0.0032 & -0.0002 & 0.0017 & -0.0001 & 0.0000 & 0.0000 & -0.0000 & 0.0000 & 0 & 0 & 0 \\ 1.0000 & 0.1187 & 0.0141 & 0.0017 & 0.0236 & 0.0028 & 0.0003 & 0.0006 & 0.0001 & 0.0000 & 0 & dt & 0 \\ 1.0000 & 0.1116 & 0.0125 & 0.0014 & 0.0206 & 0.0023 & 0.0003 & 0.0004 & 0.0000 & 0.0000 & dt & 0 & 0 \\ 1.0000 & -0.1238 & 0.0153 & -0.0019 & -0.1288 & 0.0160 & -0.0020 & 0.0166 & -0.0021 & -0.0021 & dt & 0 & 0 \\ 1.0000 & -0.0869 & 0.0076 & -0.0007 & -0.0011 & 0.0001 & -0.0000 & 0.0000 & -0.0000 & -0.0000 & 0 & 0 & dt \end{array}$$

$$T = [\text{day285} \quad \text{day85} \quad \text{day282} \quad \text{day78}]$$

Use above model, we can obtain our local dynamic geoid model as listed in table 6.8:

Table 6.8 Kalman Filter model for day 285, 85, 282, and 78

Model for Day285, 2000:

$$0.4375 - 0.9760 Y - 10.2147 Y^2 + 116.8969 Y^3 + 0.7893 X - 23.0049 X*Y + 410.0493 X*Y^2 - 92.9596 X^2 - 667.2942 X^2*Y - 762.7857 X^3 + 0.0000871 vm + 0.0004464 v8 + 0.0003595 v11$$

Model for Day85, 2001:

$$0.4375 - 0.9703 Y - 10.2144 Y^2 + 116.8971 Y^3 + 0.7867 X - 23.0057 X*Y + 410.0494 X*Y^2 - 92.9596 X^2 - 667.2941 X^2*Y - 762.7857 X^3 + 0.0000152 vm + 0.0004699 v8 + 0.0003689 v11$$

Model for Day282, 2001:

0.4375 -0.9774 Y-10.2143 Y^2+116.8969 Y^3+0.7880 X-23.0073 X*Y+410.0497 X*Y^2-92.9586 X^2-
667.2942 X^2*Y-762.7856 X^3+0.0000111 vm+0.0006231 v8+0.0004561 v11

Model for Day78, 2002:

0.4375 -0.9898 Y-10.2148 Y^2+116.8965 Y^3+0.7873 X-23.0089 X*Y+410.0500 X*Y^2-92.9566 X^2-
667.2943 X^2*Y-762.7854 X^3-0.0000111 vm+0.0007595 v8+0.0005987 v11

6.8 Prediction of Geoidal Heights

Once a best-fitting polynomial has been defined to represent the shape of geoid for a certain area by statistical trend analysis technique, it may be used to predict geoidal heights and deflections of the vertical (N_i) at unknown points or the future time geoid in modeling area. In this particular example, the predicted values at unknown points are calculated by Eq. 6.26 and Eq. 6.27.

$$H_{GPS_k} - N_{99_k} - h_{leveling_k} = \sum_{i,j} a_i x_k^j y_k^{n-j} \quad 6.26$$

where coefficients are from Tables 6.4 to 6.7 depending on the model chosen. The position of new points refers to the local xy-system.

The variance of the predicted geoidal height values is evaluated by Eq. 6.19

$$\sigma_{\Delta V}^2 = F^T D_{\Delta\Delta} F$$

$$D_{\Delta\Delta} = \hat{\sigma}^2 Q_{XX} = \sigma^2 N^{-1}$$

where $\hat{\sigma}^2$ is from Eq. 6.20. $D_{\Delta\Delta}$ is the variance-covariance matrix of the estimated unknowns in the trend functions.

To assess the validity of the predicted relative geoidal heights by the best-fitting polynomial, the NGS GEOID99 Model and local geoid model are used again to predict the relative geoidal heights for these 11 network points. They are listed in Table 6.4 to Table 6.7 (see also Figure 6.10).

A comparison between predicted and observed geoid heights show that the relative geoidal height models for each observed period are significantly different, and they differ in both magnitudes and signs. The maximum differences between them are shown in Table 6.4 to Table 6.7. The standard deviations are between 0.002 to 0.050 meters which is apparently far beyond the noise level 1 cm.

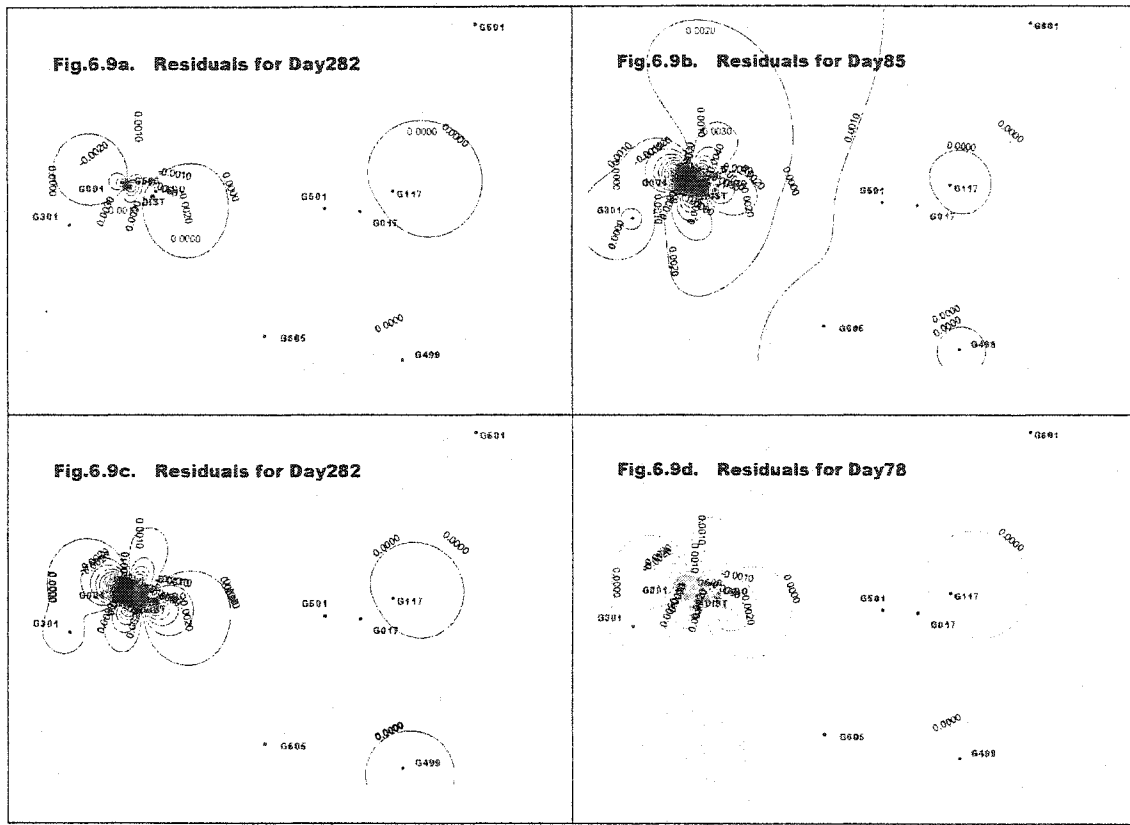


Fig. 6.10 Polynomial Fitting Residuals for Each Observed Day

After adding geoid corrections from GEOID99 and local model, we find that they didn't agree well with the points established by geodetic leveling, therefore, we conclude that the relative geoidal heights predicted by the best-fitting polynomial are not sufficiently accurate for engineering applications.

To overcome this problem, we ran a vertical network adjustment of the GPS network by holding fixed each orthometric height obtained from geodetic leveling. These three points are IADO, G017, and G506

which were selected to be uniformly distributed along the project in east and west direction, where there is the biggest geoid change in direction. The differences between the relative geoidal heights predicted by GEOID99 and local geoid model are listed in Table 6.4 to Table 6.7.

Using the Kalman model we can predict geoid correction before each observation time and filter it after each observation. The main advantage of the Kalman Filter method is that the procedure works in progressive steps. We can predict the geoid correction for the new points at any time before observation time. If later we get observations for some points we can update the local geoid. The Table 6.9 shows all the predicted values for each point before observation time and filtered value after the observation. The results show that all the predicted values agree well with the actual geoid values of the point. All of the residuals are less than 3cm and the maximal standard deviation is 0.018m, which is better than our expected value 1 to 3 cm and third order leveling accuracy.

An added advantage is that there is no systematic bias between the geoidal heights predicted by GEOID99 and local geoid model. This shows that the relative geoidal heights predicted by GEOID99 and local

Table 6.9 The Residuals of Kalman Filter Predicted and Filtered Value

Station	2000.285		2001.85		2001.282		2002.78	
		Filter	Predict	Filter	Predict	Filter	Predict	Filter
IADOT		0.000	-0.014	-0.003	-0.003	-0.002	-0.002	0.001
DIST		0.001	-0.013	-0.002	-0.019	-0.018	-0.022	-0.019
G601		0.000	-0.013	0.000	0.008	0.007	0.012	0.012
G605		0.000	0.006	0.018	0.023	0.024	0.019	0.021
G301		0.000	0.007	0.018	-0.008	-0.006	-0.014	-0.010
G117		0.000	-0.010	0.002	0.018	0.018	-0.007	-0.006
G501		0.000	-0.018	-0.006	-0.023	-0.022	-0.001	0.000
G001		0.003	0.004	0.000	0.029	0.000	0.021	0.000
G506		-0.005	-0.026	-0.015	0.009	0.010	0.001	0.004
G499		0.000	-0.024	-0.012	-0.012	-0.012	-0.004	-0.004
G017		0.000	0.001	0.000	0.018	0.000	0.024	0.000
St.d		0.002	0.015	0.010	0.018	0.014	0.015	0.011

geoid polynomial model are not accurate enough for the project area.

However, the Kalman model is more accurate than those predicted by the best-fitting polynomial for the area, and satisfy engineering application requirements.

6.9 Local GEOID Model Analyses Conclusions

The principles and applications of modeling the geoid locally using the technique of statistical trend analysis and Kalman Filter combination have been described in this research. In geodetic computation, geoid parameters are needed for coordinate transformation and for reducing surveying measurements made on the surface of the earth to an appropriately chosen reference frame, for instance, the surface of a reference ellipsoid or a projection plane, etc.

There are a variety of methods for geoid modeling. For engineering surveys limited to a relatively small area where a sufficient number of geoidal heights are known from a combined GPS and geodetic leveling technique, the surface fitting technique and Kalman filter method are proposed. The surface fitting technique and Kalman Filter combination are able to provide a local geoid with high precision.

From the practical application of the technique in a real world project, some conclusions can be drawn which are of interest to practicing surveyors or engineers. First, contrary to common belief, the given example shows that even a well-defined polynomial and Kalman Filter model, which represents the shape of the geoid locally at known data points that satisfy all the statistical criteria imposed, may not necessarily be able to predict geoidal parameters in the area with high

accuracy. The results may be acceptable for raw geodetic observation data reduction purposes in ordinary engineering projects, but for engineering projects requiring high precision, the quality of the estimated geoid parameters should be checked with other independent methods before being used. The technique of surface fitting using polynomials and Kalman Filter should, therefore, be used with discretion. It may produce better results when a different type of base function is selected to approximate the geoid. Such an investigation is left to the interested reader. When a mixed GPS and geodetic leveling survey is run in the project area, a reliable approach to obtain orthometric heights at points, with sufficient accuracy for lower order vertical control applications not done by geodetic leveling but by GPS, is to perform a vertical adjustment of the GPS network by holding a few or all available orthometric heights fixed. The fixed points should, however, be distributed evenly through the project area in order to minimize the effects of irregularity of the geoid. Finally, where the NGS GEOID99 model has proven to perform well at known points in a specific project, it may be used to predict parameters at unknown stations in the project area for certain applications. But for local and high accurate engineering applications, the GEOID96 or GEOID99 or even a later newer version, obviously could not meet the requirements.

For instance, in the example given here, by combining GEOID99 and local polynomials model, we can get a good fitness for the each observed time. But in practice, we cannot observe all local points each time we need the model every time. Given time, a geoid model of required accuracy can only be developed by combining GEOID99, region, local polynomial and Kalman Filter model. Such a combination produced the results that are accurate enough for engineering applications at both known and unknown data points in observed time and unobserved time or both.

CHAPTER 7 RESULTS AND CONCLUSIONS

This research has developed a methodology at both theoretical and practical levels, for efficiently and accurately determining sub-decimeter level region geoid and centimeter level local geoid to meet region surveying and local engineering application requirements. This research also provides a highly accurate static DGPS network data pre-processing, post-processing and adjustment method and procedure for a large GPS network like the state level HRAN project. The research also developed a very efficient and accurate methodology to join soil coverages in ARE/INFO.

The methodology developed in this research includes the following procedures and results:

1. Developed theories of designing and establishing a HARN project for the state of Iowa. A total of 181 GPS stations, at least two sessions at each station has been pre and post processed. The research also analyzed GPS errors to fix cycle slips and their effects, determined observation rejection and re-observation stations. After fixing all cycle

slips, in the Iowa HRAN project, we have obtained a better than 1.5cm absolute accuracy for 95% of the stations, and all stations have a 0.5 ppm average relative accuracy.

2. Developed theories of GPS network adjustment and adjusted Iowa HARN project, and analyzed adjustment results. A total of 167 GPS stations in Iowa and around Iowa have been included in the adjustment. Seven A-order GPS point were selected as reference points. There are a total of 861 vectors, 3444 observations were adjusted. For the Iowa HARN project, we have an accuracy at the major axis of 0.0196 m, and minor axis 0.0097 m at 95%.

3. Accurately evaluated the GEOID96 model for the state of Iowa. By GEOID96 model and 3D adjustment, a vertical height network has been developed at an average accuracy of 0.091m. This research established an orthometric height network from HARN GPS project. This research not only assessed and developed the national geoid model for Iowa, but also provided diagnostic information on the GPS network and level network accuracies, and indicated future survey requirements for Iowa.

4. After evaluating GEOID96 and GEOID99, a more accurate and suitable geoid model has been established in Iowa using both GPS and geodetic leveling information through out the whole of Iowa. This new

Iowa regional geoid model improved the accuracy from a sub-decimeter 10~20 centimeter to 5~10 centimeter. The new regional geoid developed by this research was combined with the Iowa HARN to better serve both horizontal and vertical control needs of surveyors, and to obtain the maximum economic benefit from the existing Iowa HARN project.

4. During the past three years, four sets of data have been collected at 11 points in Story County. All these data has been processed by DGPS with precise ephemeris. This research developed a methodology to establish a high accuracy local geoid model for a local area to support engineering applications. The methodology has been test by this real data. The prediction gives a better than third order leveling accuracy and 1.5 cm standard deviation.

5. In order to overcome problems in GIS edge match and associated "database join", this research develops a new algorithm, which is called Section Match to PLSS (SMP). It can not only match normal section to PLSS, but also completely match river boundaries to PLSS. Even for large offsets from PLSS, which sometimes measures up to 200 meters, the new method gets very good results.

Through the theory and practical projects in this research, some conclusions and recommendations can be drawn:

1. There are a variety of methods for geoid modeling. For engineering surveying needs limited to a relatively small area where a sufficient number of geoidal heights are known from a combined GPS and geodetic leveling, the surface fitting technique is considered to be able to provide a local geoid with high precision.

2. Errors in GPS heights and geodetic leveling height data will obviously spread into the regional and local high accurate geoid models. These errors can result from observation uncertainties from both GPS and geodetic leveling; coordinate determination, adjustment method, and data preprocessing and post processing.

3. With the wide use of GPS in engineering, a precise geoid can be useful, especially for determining orthometric heights from GPS-derived ellipsoidal heights.

4. It should always be kept in mind that, there is only one real local geoid in the real world, even if it is changing in time. At the moment the knowledge of this unique geoid model has some uncertainties and errors that are big, but with more and more points added to the model and local geoid determination network, the local geoid model given in this research will provide better and better estimates with time.

5. In large areas, a national model, like GEOID99 or GEOID96 will give the best geoid height accuracies; with more and more GPS point adding to the model the accuracy will be higher.

6. Numerical evaluation showed that national geoid model could only produce sub-decimeter geoid accuracies in a local region like Iowa.

7. In both regional and local geoid modeling, it is necessary to consider all possible sources of information. Adding more and more GPS and levelling will improve regional geoid modeling.

8. The biases caused by interpolation and smoothing, together with observation noise, should be investigated, specified, carefully estimated and taken into consideration in the entire geoid modeling process.

9. Geoid modeling methodology should not be developed merely to ensure that the geoid values agree with GPS/levelling geoid correction values, as there are sources of errors that are often not accounted for in either the GPS heights or the geodetic leveling heights or both. When we consider highly accurate local geoid modeling at cm-precision we often focus and pay too much to the problem of data availability. But this is not the only problem in achieving cm-precision. In my opinion, a better high-resolution model is also necessary to

remove some errors. In this research we have discussed the errors and their meaning and effect.

APPENDIX A GIS COVERAGE (GEODATABASE)

EDGE MATCH

A.1 Introduction

Geographic Information Systems (GIS) use a variety of different spatial reference points, but they are of little value if they are not reliable. This is one reason why the National Geodetic Survey (NGS) completed the High Accuracy Reference Networks (HARNS) for every state. The HARN will benefit the surveying and engineering communities in U.S. In the long term, it will certainly be important in the land surveying as the framework for the geodetic network in the U.S.

All maps are created from certain datum and projections, which are directly related to a reference frame. Before If GIS uses these maps they must be transformed, converted and digitized. To do so a reliable network is needed.

The traditional soil survey reports developed by the United States National Cooperative Soil Survey Program are the product of the most comprehensive natural resource inventory in the world. The format of the report, consisting of a bound document with three major sections: Text, Tables and Maps, has been unchanged for the past 40 to 50 years. Among these three major sections, maps are the basic section, they are the media and carrier of information. Without accurate and rightly formatted maps, it is impossible to develop a high accurate, efficient digit soil information system.

There is a soil survey for most of the 3,000 plus counties in the United States. However, the scale, base map, and classification system used varies from county to county. The current emphasis is to bring consistency to the soil survey across political boundaries using the knowledge gained from the past 100 years (Indorante et al. 1996). Many States are trying to create a digital soil database and mapping system.

A.2 IOWA Soil Project

With funding from the Iowa DNR (Iowa Department of Natural Resources Geological Survey Bureau), we started Iowa soils project, an interactive digital soil survey with a wish list of features that included:

- Pure database capability
- Links between maps tables, and text
- Map manipulation via database manipulation
- Hyper-links for graphics and text
- Capability of viewing multiple map "sheets" as one image
- Easy to Use.

Tabular data with a few exceptions is available, which a user would find in a soil survey report. It contains 189 items (see table A. 1),

Table A.1 Tabular Data of Soil Coverage

Sms	Sml	Sscsoils	Soilname	Lcc	Erimalnd	Longalnd	Csr	Cornfld	Soybnfld
11B	0011B1	KS0011	VESSEK-MODAWAY COMPLEX	2W	D1	D4	65	126	
24C	0024C1	IA0142	SEELY	2E	S	D1	60	128	
24C2	0024C2	IA0502	SEELY	2E	S	D1	58	128	
24D	0024D1	IA0142	SEELY	2E	S	S2	50	119	
24D2	0024D2	IA0502	SEELY	2E	S	S2	48	119	
24E	0024E1	IA0142	SEELY	4E	L	S1	40	102	
24E2	0024E2	IA0502	SEELY	4E	L	S1	38	98	
24F2	0024F2	IA0502	SEELY	6E	L	O	18	0	
51	0051A0	IA0119	VESSEK	2W	D1	D1	70	130	
69C	0069C1	IA0011	CLEARFIELD	2W	S	S1	50	112	
69C2	0069C2	IA0517	CLEARFIELD	2W	S	S1	41	108	
92D2	0092D2	KS0091	SEELY-ADAIR COMPLEX	4E	S	S1	25	95	
93E2	0093E2	KS0091	SEELY-ADAIR COMPLEX	6E	L	O	10	0	
179C2	0179C2	IA0520	DICKINSON	3E	S	S1	25	66	
179D2	0179D2	IA0520	DICKINSON	4E	L	S2	14	57	
179D	0179D1	IA0154	GARA	4E	L	S2	45	110	
179D2	0179D2	IA0562	GARA	4E	L	S2	43	106	
179E2	0179E2	IA0562	GARA	6E	L	O	21	0	
179F2	0179F2	IA0562	GARA	6E	L	O	14	0	
182C2	0182C2	IA0551	ADAIR	2E	S	S2	10	82	
182D2	0182D2	IA0551	ADAIR	4E	L	S1	15	71	
222C	0222C1	IA0141	CLARINDA	4W	S	S2	10	82	
222C2	0222C2	IA0544	CLARINDA	4W	S	S1	25	72	
222D2	0222D2	IA0544	CLARINDA	4E	S	O	10	61	
271B	0271B1	IA0140	OLNEY	2E	D	D1	72	137	
271C	0271C1	IA0140	OLNEY	2E	S	D1	57	132	
287B	0287B1	KS0287	ROCK-COLO-ELY COMPLEX	2W	D5	S1	70	134	

which give a detailed description of soil coverage.

Original Source Information

Copies of individual soil survey map sheets, which are of is 1:15,840 and 1:20,000 scale, were scanned with an Optronics camera at 200 DPI (Ground resolution 2 meters). Map sheets were originally drawn on black and white rectified aerial photos (Fig. A.1). These aerial photos were not orthorectified and thus soil lines contain terrain distortions inherited from the imagery.



Fig. A.1 Rectified Aerial Photography Base Maps

The map projection is Universal Transverse Mercator (UTM), Zone 15 DISTANCE UNITS: Meters DATUM: NAD27. In order to digitize the map, an optical scanning device was used to digitize the recompiled soil information on a section by section basis and geo-referenced.

Scanned images were edited with MES (Map Editing Software). Map sheets were clipped to individual sections having a raster resolution of 320 x 200 pixels (typical ground resolution 5 x 8 meters). After digitizing, raster data was converted to vector in Arc/Info format.

Soil data was transformed to the UTM coordinate system using registration tic files from Iowa DNR NRGIS county Public Land Survey System (PLSS) files.

PLSS coverage's were digitized from 24k USGS topographic maps having 22 meter RMS accuracy.

If soil boundary lines are to be digitized and overlaid with other geo-referenced data layers, they need to be consistent with the original published survey. The spatial accuracy of the line work in a soil survey is a function of three characteristics: scale, photographic base and topographic relief.

A.3 Section Match Method

Iowa has 99 counties, containing 1671 townships and more than 200,000 sections. Because of the map scale, scanner size, and other factors, all original digital coverage is on section by section basis (Fig.A.2). In order to join the base coverages of sections together to get township & county size soils coverage, we must match section base

coverage to PLSS coverage thus obtaining a standard boundary section coverage.

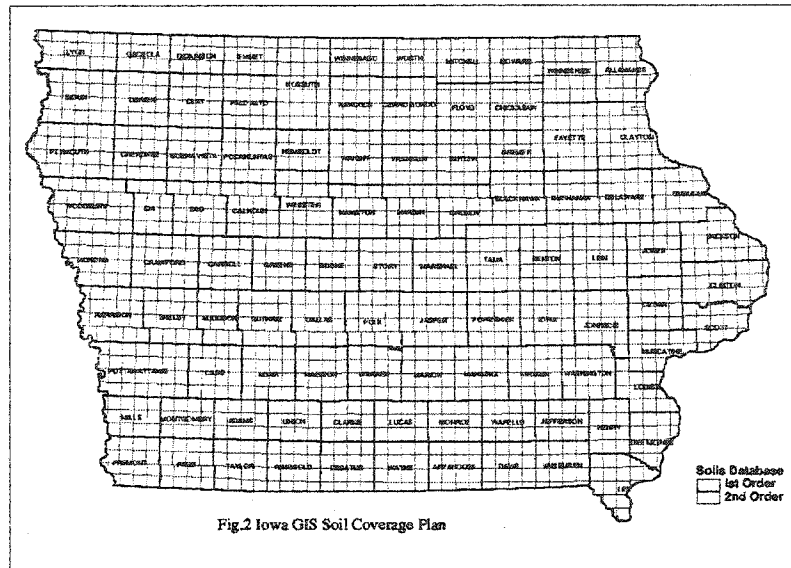


Fig. A.2 Iowa Soil GIS Coverage Map

Arc/Info offers a basic adjustment function, called "adjust", to do this. For a single coverage, you can adjust it to a snapcoverage manually in Arcedit. But for more than 200,000 coverages, it will be impossible to adjust them one by one, manually. If one coverage takes one hour to adjust, 200,000 coverages will take about 70 GIS technicians working one year to finish the adjustments, ignoring any mistakes during processing and final processing, such as final coverage joining and final table joining. This is only the first step in creating GIS soils database on township or county size base. After those line, point and polygon match, we must:

APPEND -- combines features from two or more coverages into a single output coverage. This does not calculate new topological relationships between the resulting features.

Before appending all section coverages to a township or county size coverage, we must match each section coverage to PLSS to get a common boundary. There is a sample method with Arc/Info Macro, developed by Iowa DNR to do this. But this method with Macro, AML (Arc/Info Macro Language) can only do a standard section match. It uses tic match method and distance search algorithm. It can match less than a 60% standard rectangle sections, and less than 10% of township (see Fig. A.6 Lee County, Section & Township Match to PLS Status). The reasons are (Fig.A.3 Comparison between PLS Match Method):

- It only uses coverage corner tic split algorithm (CTSA).
- It considers all section boundaries as straight line
- It assumes that all section coverages have same tic points
- It assumes either there are no node points at corner and corner lines intersect at 90 degree

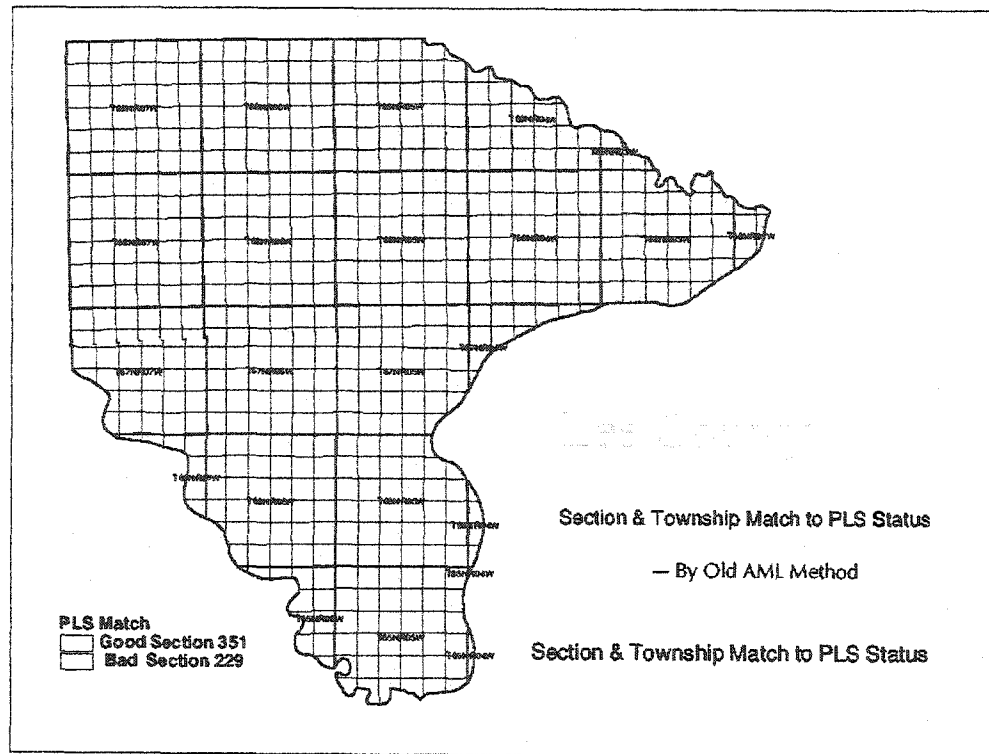


Fig. A.3 Comparison between PLS Match Method

To overcome these problems, we developed a new algorithm, which is called Section Match to PLSS (SMP). It can completely match river boundary to PLSS. Even for some very bad and large offsets from PLSS, which can sometime be up to 200 meters, the new method gets very good results (see Fig.A.4 to Fig. A.6) (Fig.A.7 Match River Section with New Method).

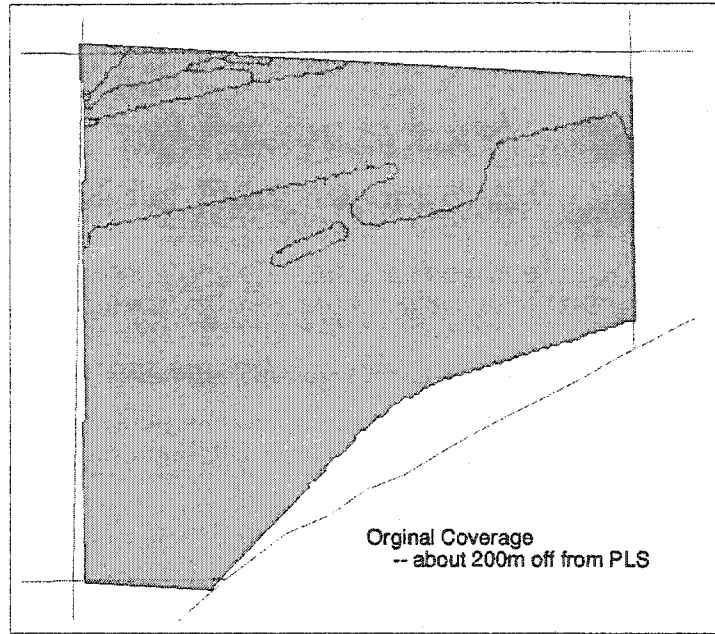


Fig. A.4 Original Coverage

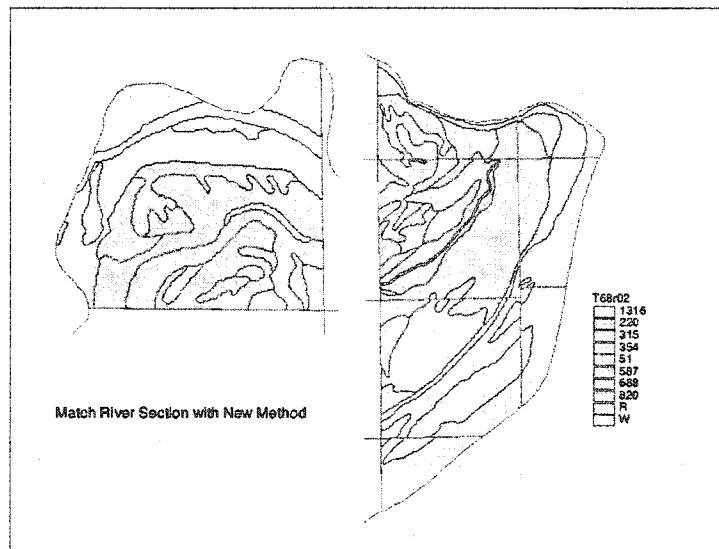


Fig. A.5 Match River Section with New Method

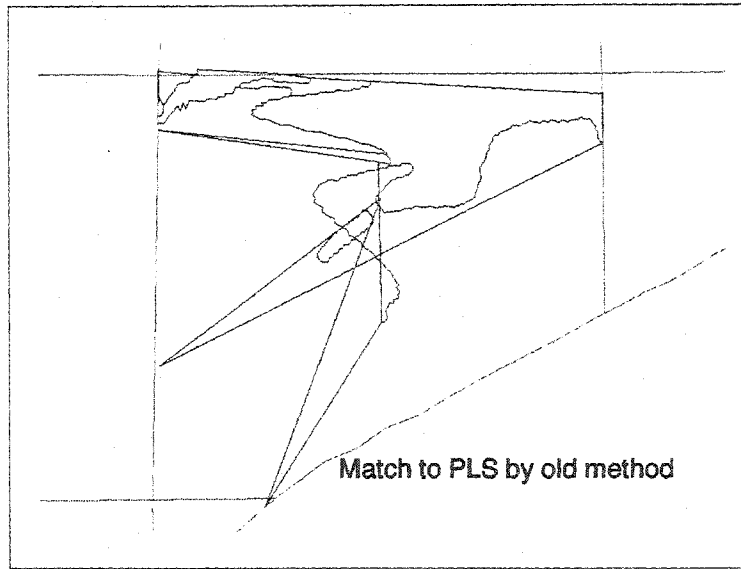


Fig. A.6 Match to PLS by Old Method

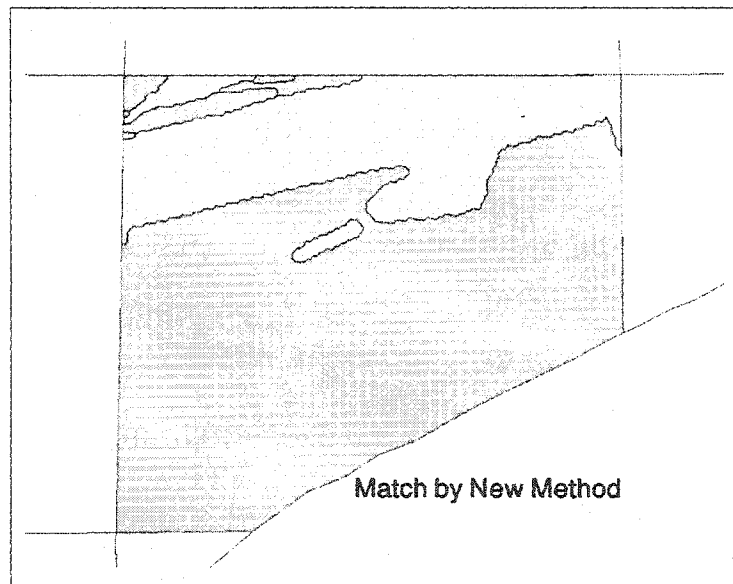


Fig. A.7 Match By New Method

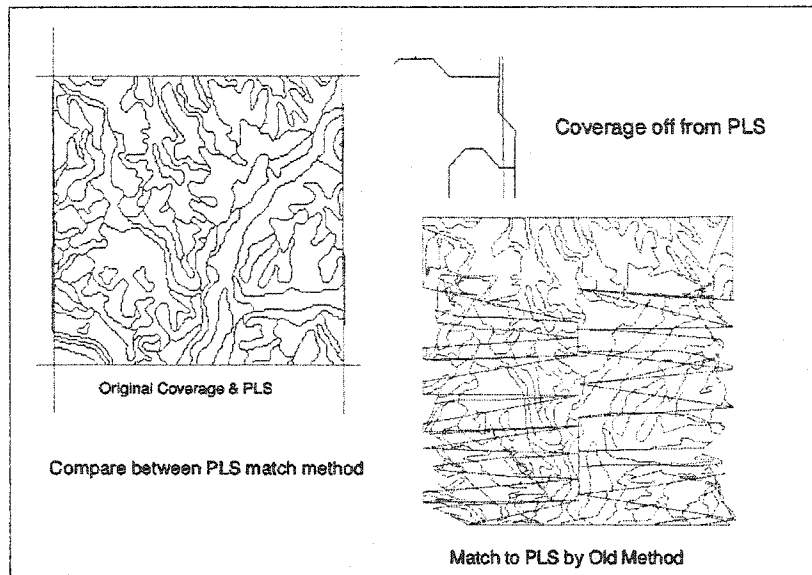


Fig.A.8 Comparison between New and Old PLS Match Method

The basic principles of this in new algorithms (SMP) are:

Densify PLS ARC (DENSIFYARC)

Densifying PLS arc to create a new snap coverage for section adjustment adds vertices to arcs (in a coverage) at a specified interval and alternately splits the arcs at each new vertex. Section snap can be separated into three links (Fig.7.9 Section Match to PLS Principle):

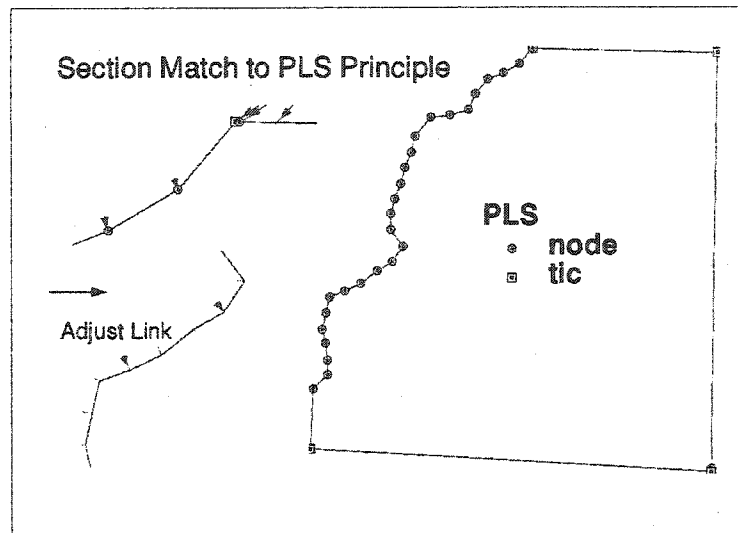


Fig. A.9 Section Match to PLS Principle

--- Arc to Arc, Node to Node, Arc to Node

DENSIFYARC is often used for feature snapping of arcs to arcs prior to executing SNAPCOVERAGE in ARCEDIT. Since arc-to-arc snapping looks for vertices which are within the snap distance of the coverage's of another arcs, the two coverages should be of the same density in the number of vertices along each arc. Also, it is often useful for the arcs in the coverage to be denser than the arcs in the snap cover to which they will be snapped. DENSIFYARC will be useful in these situations.

Corner Distinguish Algorithm (CDA)

This algorithm offers a true corner judgment for each section boundary node and vertices, or split section boundary at nearest

corner when section boundaries without true node or vertices at corner, and links this point to PLSS corner or TIC points of PLSS.

Direction Split Coverage (DSC)

To completely match section coverage to PLSS, the key is that we need to split section boundary to get the same number or more nodes and vertices than in the PLSS coverage. This DSC method can split the section coverage boundary arc in the correct direction, it will split the arc when split location is a node or vertices, and it can also find coordinates for each new split point and link it to corresponding node or vertices at PLSS (Fig.A.9 Section Match to PLS Principle).

Manual Interactive Function (MIF)

For some special situations we need to fix or judge section coverages or boundaries manually. This method offers an Arc/Info AML to help the user modify the decision made by the algorithm itself.

The new method SMP will match more than 95% of section coverage to PLSS boundary completely (Fig.A.8 Match River Section with new Method), and do all the following procedures in Arc/Info automatically:

IDEDIT -- updates User-IDs in the coverage after modification in a feature attribute table. If User-IDs are modified in a feature of the attribute table, IDEDIT should be used to update the User-IDs in the coordinate files of the coverage's. In IDEDIT, the feature attribute table is used to determine the correct User-ID for each feature in the coverage. This value is recorded in all places where the feature User-ID is stored.

INDEX -- creates a spatial index for a coverage which improves the function of any operation that retrieves coverage features by location. INDEX creates additional files for a coverage which require 32 bytes per feature. This typically increases the size of a coverage by about 8 to 10 percent.

The purpose of an index is to improve drawing, and spatial selection and IDENTIFY performance in ARCPLOT. An index is particularly useful for large coverages (e.g., with over 10,000 arcs) and for map libraries where a drawing window is used to zoom in on a portion of the coverage or map library area. Drawing and selection speed can be increased by up to 40 times. Indexes are not an integral

part of a coverage. They are created to improve spatial selection and drawing for ARCPLOT query and drawing functions.

FREQUENCY -- produces a list of the unique code occurrences and their frequency for a specified set of items in an INFO data file. Optionally, summary items may be totaled for each unique code combination (e.g., the total AREA for unique combinations of ZONING and LAND-USE).

FREQUENCY may be used in the initial examination of a data set to see what occurs in the data set, or as an aid in classification to help identify cases that are not really different. FREQUENCY can also be used to pick out unusual cases for closer examination. Output from FREQUENCY is of two possible forms. In all cases, a table of unique occurrences is generated. If a case item is specified, then a case item containing the record number of the unique case pertaining to that particular record is added to the INFO data file being examined. This new item can be used to relate the two INFO files.

DISSOLVE -- merges adjacent polygons, lines or regions which have the same value for a specified item.

DISSOLVE eliminates nodes or arcs between adjacent lines or polygons containing equal values for the dissolve item. With the POLY or NET option, if merged polygons contain label points, one of the points is preserved in the output coverage. If the POLY option has been specified, dangling arcs of any length are removed; pseudo nodes are also removed, unless they are the only node in a polygon (e.g., an island or donut). With the LINE option, DISSOLVE eliminates nodes between adjacent arcs containing equal values for the dissolve item. The new cover-ID of the merged arc will be the lowest cover-ID of the combined arcs. If a resulting arc reaches the 500 vertex per arc size limit, a pseudo node is added and a new arc is begun. With the NET option, arcs are joined as units and before two arcs are joined, the total number of vertices is compared against the 500 vertex limit. If greater than 500, the two arcs are not joined.

DISSOLVE is used to create a simplified coverage from one which is more complex. While the input coverage may contain information concerning many feature attributes, the output coverage contains information only about the dissolve item.

ELIMINATE -- merges selected polygons with neighboring polygons that have the largest shared border between them, or that have the largest area. With the **LINE** option, **ELIMINATE** merges selected arcs separated by pseudo nodes into single arcs.

ELIMINATE is used most often to remove sliver polygons created in an overlay of two coverages where the arc overlap is similar, but not exact.

ELIMINATE reduces the number of polygons or lines in a coverage by merging selected features with one of their neighbors.

For the **POLY** option with **BORDER**, the longest arc of each selected polygon is eliminated. For the **POLY** option with **AREA**, the largest adjacent unselected polygon is located and the arc between it and the selected polygon is eliminated. Label points are eliminated when a whole polygon is eliminated. Topology is then updated. All polygons in the output coverage will have only one label point. For the **LINE** option, an arc to be eliminated is merged with a connecting arc that preceded it in the processing order of the **ELIMINATE** command. The resultant arc will assume the direction of the arc, the eliminated arc merges into.

EXPORT -- Export an ARC/INFO coverage to an interchange file.

An ARC/INFO interchange file can be used to transport coverages, INFO data files, text files such as AML macros, and other ARC/INFO files between various machine types. An interchange file contains all coverage information and appropriate INFO data file information in a fixed-length ASCII format. The EXPORT command is used to create an interchange file. The interchange file can be transferred by tape to another machine where IMPORT can be used to re-create the coverage, text, INFO file, or other data type.

REBUILD TOPOLOGY -- creates or updates a feature attribute table for a coverage. Topology is created by identifying the set of arcs which define each polygon and the internal numbers of polygons which are to the left and right sides of each arc. From-nodes and to-nodes are used to determine the left/right orientation for each arc.

APPENDIX B

Iowa Soil Project Final Processing

There are some final processing procedures after GIS soil data match and merge before it could be used as a GIS soil database:

Join info file for soils coverages

- joinitem ispaidxx (county number) file label to SMS *.pat
- load ispaidxx.dbf file
- in tables or info:
 - dbaseifc ispaidxx.dbf ispaidxx.pat 1
 - convert it to info file ispaidxx.pat 1
- add new item label in ispaidxx.pat 1
- copy SMS to label
- joinitem

Replace Tic Points

Delete all tic points, replace with new tics from county plss coverage

- All the soils polygon coverages need arc attributes, so run following arc build for each soil coverage
 - arc build s99n41w line
- after building xxx line command, the IDs for the soil arc attributes need to be recalculated, then have idedit run
 - arc>tables
 - sel s99n41w.aat
 - calc s99n41w-id = s99n41w#
 - q
 - arc>idedit s99n41w line
- coverages need to have their spatial indexes created
 - arc>index s99n41w all
- frequency needs to be run on each soil polygon coverage
 - arc> frequency s99n41w.pat s99n41w.freq
 - label
 - end

```

y
area
end
y
y

```

For the Drains coverages

- append all data to one county, name drainsxx (county number)
- joinitem *.aat drain *.aat attribute attribute
in table or info do:
 dbaseinfo DRAINS.dbf DRAINSxx.pat1
in arc
 JOINITEM drainsxx.aat drainsxx.aat1 drainsxx.aat attribute attribute
- in tables or info
 calc IDs
 tables
 sel drain**.aat
 calc calc DRAINS**id = DRAINS**#
 quit
 run idedit on line
 idedit DRAINS** line
 index DRAINS** all
- delete tic , replace with PLSS tics
Spots:
append all data to one coverage, spotsxx (county number)
joinitem *.pat symbol *.pat attribute attribute
calc IDs in tables, idedit on point
index
delete tics, replace with PLSS tics

For the Water coverages

- append all data to one coverage , with name waterxx (county number)
- joinitem *.pat symbol *.pat attribute attribute
- delete tics , replace with PLSS tics
- calc IDs in tables, idedit on poly
 tables
 sel Water**.pat
 calc Water**-id = Water**#
 q
 build line
 idedit Water** line
 index Water** all

AML Program for Soil Project

Many AML programs have been written this project. Followings are some main AML program. Because of size, these couldn't list all AML programs. If readers are interested some detail, please feel free to contact me.

1. State boundary match AML

```

/*=====
/* This AML works for State boundary match
/*   June 20, 1998 by Ruifeng Xi
/*=====

&watch temp.wat
&echo &on

/* snap coverage
&sv .work_pls id

/* the work coverage
&sv .work_section idown_alb

/* build %.work_section% line
/* build %.work_section%

/* create vertex for snap coverage
/* densifyarc %.work_pls% # 2000 arc

/* computer node coordinates for all arc node
ae
display 9999
edit %.work_pls%
mape %.work_pls%
de all
draw

/* Do loop to get the coordinates of both nodes on all the arcs of
/* the external polygon and matched polygon.

&sv fileunit6 := [ open bound_xy.dat openstat6 -w ]
&if %openstat6% ne 0 &then &return &inform Could not open bound_xy.dat file.

ef arc
sel all
&sv arc_number = [show number select]

&sv total_num = [ calc %arc_number% * 2 ]
&s writestat6 := [write %fileunit6% %total_num% ]

```

```

&do i = 1 &to %arc_number% &by 1
  /* Get record number of ith arc.
  &sv arc_id = [show select %i%]

  /* Get coords of nodes of ith arc.
  &sv arc_coord = [show arc %arc_id% nodes]
  /* Set values of coords of nodes.
  &sv n1x = [extract 1 %arc_coord%]
  &sv n1y = [extract 2 %arc_coord%]
  &sv n2x = [extract 3 %arc_coord%]
  &sv n2y = [extract 4 %arc_coord%]

  &s writestat6 := [write %fileunit6% %n1x% ]
  &s writestat6 := [write %fileunit6% %n1y% ]
  &s writestat6 := [write %fileunit6% %n2x% ]
  &s writestat6 := [write %fileunit6% %n2y% ]

&end

&s closestat6 := [close %fileunit6%]

&severity &warning &ignore
&severity &error &ignore

edit %.work_section%
draw arcs node link tic

/* Set backcover as county section lines from appropriate directory.
&sv snap_pls %.work_pls%

backcov %snap_pls% 4
backenv arc node

draw

/* write all arc nodes to arc_coordi.dat for late link
/* =====
/* open a file arc_coord.dat for store arc coordinates

/* &if %.sc3% ne 0 &then
/*   &sv .sc3 := [close %sf3%]

&sv .sf3 := [ open arc_coord.dat .so3 -w ]
&if %.so3% ne 0 &then
  &return &inform Could not open arc_coord.dat file.

&sv total_arc = 0

ef arc
sel lpoly# = 1 or rpoly# = 1 or lpoly# = 0 or rpoly# = 0

```

```

resel lpoly# ne 1 or rpoly# ne 1

&sv arc_number = [show number select]

&do i = 1 &to %arc_number% &by 1
  &sv sp_id = [show select %i% ]
  &sv temp_xy = [ show arc %sp_id% node ]
  &sv temp_x1 = [extract 1 %temp_xy% ]
  &sv temp_y1 = [extract 2 %temp_xy% ]
  &sv temp_x2 = [extract 3 %temp_xy% ]
  &sv temp_y2 = [extract 4 %temp_xy% ]

  /* find if the point exist and is not tic point
  &sv judge = 0
  &do j = 1 &to %total_arc% &by 1
    &sv dx = [ abs [ calc %temp_x1% - [ value cnodex%j% ] ] ]
    &sv dy = [ abs [ calc %temp_y1% - [ value cnodey%j% ] ] ]
    &if %dx% < 1 and %dy% < 1 &then
      &sv judge = %judge% + 1
    &end

  &if %judge% = 0 &then
    &do
      &sv total_arc = %total_arc% + 1
      &sv cnodex%total_arc% = %temp_x1%
      &sv cnodey%total_arc% = %temp_y1%
      &s .sw3 := [write %.sf3% %temp_x1%]
      &s .sw3 := [write %.sf3% %temp_y1%]
    &end

  &sv judge = 0
  &do j = 1 &to %total_arc% &by 1
    &sv dx = [ abs [ calc %temp_x2% - [ value cnodex%j% ] ] ]
    &sv dy = [ abs [ calc %temp_y2% - [ value cnodey%j% ] ] ]
    &if %dx% < 1 and %dy% < 1 &then
      &sv judge = %judge% + 1
    &end

  &if %judge% = 0 &then
    &do
      &sv total_arc = %total_arc% + 1
      &sv cnodex%total_arc% = %temp_x2%
      &sv cnodey%total_arc% = %temp_y2%
      &s .sw3 := [write %.sf3% %temp_x2%]
      &s .sw3 := [write %.sf3% %temp_y2%]
    &end

&end /* end for find arc_node coordinates

```

```

&s .sc3 := [close %.sf3%]

/* start to split coverage boundary recording pls nodes
/* =====

/* find pls boundary node coordinates

/* &if %.sc1% ne 0 &then
/*   &sv .sc1 = [ close %.sf1% ]

&sv .sf1 := [ open bound_xy.dat .so1 -r ]
&if %.so1% ne 0 &then
  &return &inform Could not open bound_xy.dat file.

&sv bound_num = 0
&s record_num := [read %.sf1% .sr1]
&do i = 1 &to %record_num% &by 1
  &s record1 := [read %.sf1% .sr1]
  &s record2 := [read %.sf1% .sr1]

  &sv jadge = 0
  &do j = 1 &to %bound_num%
    &sv dx = [ abs [ calc %record1% - [ value boundx%j% ] ] ]
    &sv dy = [ abs [ calc %record2% - [ value boundy%j% ] ] ]

    &if %dx% < 1 and %dy% < 1 &then
      &sv jadge = %jadge% + 1
    &end

    &if %jadge% = 0 &then
      &do
        &sv bound_num = %bound_num% + 1
        &sv boundx%bound_num% = %record1%
        &sv boundy%bound_num% = %record2%
      &end
    &end
  &end
&sv .sc1 := [close %.sf1%]

/* open a file node_coord.dat for store node coordinates
/* &if %.sc2% ne 0 &then
/*   &sv .sc2 := [close %.sf2%]

&sv .sf2 := [ open node_coord.dat .so2 -w ]
&if %.so2% ne 0 &then
  &return &inform Could not open node_coord.dat file.

&sv total_node = 0

```

```

ef arc
editd 200
&sv d_unit = 5

&do i = 1 &to %bound_num% &by 1
  sel lpoly# = 1 or rpoly# = 1 or rpoly# = 0 or lpoly# = 0
  resel lpoly# ne 1 or rpoly# ne 1

  &sv xb [ value boundx%i% ]
  &sv yb [ value boundy%i% ]

  &pushpoint 1 %xb% %yb%
  resel
  &pushpoint 1 %xb% %yb%
  split

  /* find split node coordinates
  &sv arc_number = [show number select]

  &if %arc_number% = 1 &then
    &do
      &sv sp_id = [show select 1 ]
      &sv temp_xy = [ show arc %sp_id% node ]
      &sv temp_xy1 = [extract 1 %temp_xy% ] [extract 2 %temp_xy% ]
      &sv temp_xy2 = [extract 3 %temp_xy% ] [extract 4 %temp_xy% ]
      &sv d1 [ invdistance %xb% %yb% %temp_xy1% ]
      &sv d2 [ invdistance %xb% %yb% %temp_xy2% ]

      &if %d1% < %d2% &then
        &do
          &sv px = [ extract 1 %temp_xy% ]
          &sv py = [ extract 2 %temp_xy% ]
        &end
      &else
        &do
          &sv px = [ extract 3 %temp_xy% ]
          &sv py = [ extract 4 %temp_xy% ]
        &end

      &sv arc_number = [show number select]
      &do j = 1 &to 5 &while %arc_number% = 1
        &sv d_unit = [ calc %j% * %d_unit% ]
        &sv angle = [ show arc %sp_id% orientation %px% %py% ]
        &sv dx = [ calc %d_unit% * [ cos %angle% ] ]
        &sv dy = [ calc %d_unit% * [ sin %angle% ] ]

        &sv arc_number = [show number select]
        &if %arc_number% = 1 &then
          &do

```

```

    &pushpoint 1 [ calc %px% + %dx% ] [ calc %py% + %dy% ]
    split
&end

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
    &do
        &pushpoint 1 [ calc %px% + %dx% ] [ calc %py% - %dy% ]
        split
    &end

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
    &do
        &pushpoint 1 [ calc %px% - %dx% ] [ calc %py% - %dy% ]
        split
    &end

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
    &do
        &pushpoint 1 [ calc %px% - %dx% ] [ calc %py% + %dy% ]
        split
    &end

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
    &do
        &pushpoint 1 [ calc %px% + %d_unit% ] [ calc %py% ]
        split
    &end

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
    &do
        &pushpoint 1 [ calc %px% - %d_unit% ] [ calc %py% ]
        split
    &end

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
    &do
        &pushpoint 1 [ calc %px% ] [ calc %py% - %d_unit% ]
        split
    &end

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
    &do
        &pushpoint 1 [ calc %px% ] [ calc %py% + %d_unit% ]

```

```

        split
    &end

    &end /* end for j = 1
&end /* end for &do arc_num = 1

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
    &do
        &s .sw2 := [write %.sf2% %px%]
        &s .sw2 := [write %.sf2% %py%]
        &s .sw2 := [write %.sf2% %xb%]
        &s .sw2 := [write %.sf2% %yb%]
        &sv total_node = %total_node% + 1
    &end
&else
    &do
        &do j = 1 &to 2 &by 1
            &sv sp_id%j% = [show select %j% ]
            &sv spf%j% = [ show arc [ value sp_id%j% ] FNODE# ]
            &sv spt%j% = [ show arc [ value sp_id%j% ] TNODE# ]
            &sv sp_xy = [show arc [ value sp_id%j% ] nodes]
            &sv spfx%j% = [extract 1 %sp_xy%]
            &sv spfy%j% = [extract 2 %sp_xy%]
            &sv sptx%j% = [extract 3 %sp_xy%]
            &sv spty%j% = [extract 4 %sp_xy%]
        &end

        &if %spf2% = %spt1% &then
            &do
                &sv spx = %spfx2%
                &sv spy = %spfy2%
                &sv sp_node = %spf2%
            &end
        &else
            &do
                &sv spx = %spfx1%
                &sv spy = %spfy1%
                &sv sp_node = %spf1%
            &end

            &s .sw2 := [write %.sf2% %spx%]
            &s .sw2 := [write %.sf2% %spy%]
            &s .sw2 := [write %.sf2% %xb%]
            &s .sw2 := [write %.sf2% %yb%]
            &sv total_node = %total_node% + 1
        &end
    &end

```

```

&end /* end for ( &do i = 1 &to %bound_num% &by 1 )
&s .sc2 := [close %.sf2%]

/* links arc to pls, & node to node
/* =====
/* Select all the arcs of the external polygon and generalize them.
sel lpoly# = 1 or rpoly# = 1 or rpoly# = 0 or lpoly# = 0
resel lpoly# ne 1 or rpoly# ne 1

generalize 1000

/* Set appropriate edit data for edgematching and perform the edgematching
editf link
/* Set snapcover to section line coverage in appropriate directory.
/* snapcov %.PATH%/%.COUNTY%
snapcov %snap_pls%

/* link node to node
/* -----
snapfeature node node
snapping closest 200
linkfeature node node

/* Read the coordinates of the nodes of the external polygon and use them to
/* link to the section lines.

/* link boundary node
/* -----
/* read node from boundary nodes node_coord.dat

/* &if %.sc4% ne 0 &then
/*   &sv .sc4 := [close %.sf4%]
/* &s .sf4 := [open node_coord.dat .so4 -r]
/* &if %.so4% ne 0 &then
/*   &return &inform Could not open file node_coord.dat.

&do i = 1 &to %total_node% &by 1
  &s record1 := [read %.sf4% .sr4 ]
  &s record2 := [read %.sf4% .sr4 ]
  &s record3 := [read %.sf4% .sr4 ]
  &s record4 := [read %.sf4% .sr4 ]
  &pushpoint 2 %record1% %record2%
  &pushpoint 2 %record3% %record4%
&end
&sv .sc := [ close %.sf4% ]

&pushpoint 9 0 0

```



```

/* Add links using the node coords in pushpoint.
add

/* link node to arc
snapfeature node arc
snapping closest 200
linkfeature node arc

/* Read the coordinates of the nodes of the external polygon and use them to
/* link to the section lines.

&if %.sc5% ne 0 &then
  &sv .sc5 := [close %.sf5%]

&s .sf5 := [open arc_coord.dat .so5 -read ]
&if %.so5% ne 0 &then
  &return &inform Could not open file arc_coord.dat.

&do i = 1 &to %total_arc% &by 1 &while %.sr5% ne 102
  &s record1 := [read %.sf5% .sr5 ]
  &s record2 := [read %.sf5% .sr5 ]
  &pushpoint 2 %record1% %record2%
  &pushpoint 2 %record1% %record2%

&end

&s .sc5 := [close %.sf5%]
&pushpoint 9 0 0

/* Add links using the node coords in pushpoint.
add

adjust
save

&return

```

2. Section match pls boundary AML

```

/*=====
/* smb2n1.aml -- section match pls boundary, directly split at corner
/*           Written by Ruifeng Xi
/*           GIS lab, Iowa State University
/*
/* This aml edgematches a section of soil data to the section lines
/* ( pls coverage ) for that section. The macro selects the arcs around
/* each corners of the section, find corner point, if it can't find ture
/* corner point then splits the arcs at the corner coordinates.
/* The arcs that make up the external polygon are then selected and
/* generalized to make the border of the section a set of straight lines.
/* Then the nodes on the border are edgematched to the pls and the

```

```

/* adjustments made to the coverage within 80 meters of the edge. This
/* macro is called by spt.aml. It also can be run by itself.
/*
/*
=====

/* initial environment
&station 9999

/* &echo &on
/* &watch temp.wat
/* &sv .work_section s800215et1

&severity &warning &ignore
&severity &error &ignore

&do i = 1 &to 5
  &if [ value .sc%i% ] ne 0 or [ value .so%i% ] ne 0 &then
    &sv .sc%i% := [close [value .sf%i%]]
  &end

edit %.work_section%

/* replace drain coverage tic with soil tic
&sv d_type = [ substr %.work_section% 1 1 ]
&if %d_type% = d &then
  &do
    &sv d_name = [ substr %.work_section% 2 6 ]
    &sv s_name = s%d_name%
    ef tic
    sel all
    delete
    get %s_name%
    save
  &end
draw arcs node link tic

/* Set backcover as county section lines from appropriate directory.
&sv snap_pls temp_pls
backcov %snap_pls% 4
backenv arc node
draw

/* find tic points and coordinates
editf tics

/* Uses the map extent to select the tics on the four corners of the coverage
&sv me [show mapextent]
&sv x1 [ extract 1 %me% ]
&sv y1 [ extract 2 %me% ]
&sv x2 [ extract 3 %me% ]

```

```

&sv y2 [ extract 4 %me% ]

/* Use x and y values for selection box
&pushpoint 1 %x1% %y1%
&pushpoint 1 %x2% %y2%
sel box

/* Gets the coordinates of the tics
&sv tic_number = [show number select ]
&do i = 1 &to %tic_number% &by 1
  &sv t%i% = [show select %i% ]
  &sv t%i%xy [show tic [ value t%i% ] coordinate]
  &sv t%i%x [before [ value t%i%xy ] , ]
  &sv t%i%y [after [ value t%i%xy ] , ]
&end

/* find corner number and coordinates for snapping to pls coverage
ef arc
editd 20
&sv circle_r 20

/* find corner around each tic point
&do i = 1 &to %tic_number% &by 1
  /* directly goto split
  sel lpoly# = 1 or rpoly# = 1 or lpoly# = 0 or rpoly# = 0
  resel lpoly# ne 1 or rpoly# ne 1
  &pushpoint 2 [ value t%i%xy ]
  resel
  &sv arc_id = [show select 1 ]
  &if [show arc %arc_id% npnts] > 2 &then &goto start_split

  /* directly goto split
  &sv sign = 0
  sel lpoly# = 1 or rpoly# = 1 or lpoly# = 0 or rpoly# = 0
  resel lpoly# ne 1 or rpoly# ne 1
  &sv prx = [ value t%i%x ] - %circle_r%
  &pushpoint 2 [ value t%i%xy ]
  &pushpoint 1 %prx% , [ value t%i%y ]
  resel circle passthru

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &sv arc_id = [show select 1 ]
    &if [show arc %arc_id% npnts] > 2 &then
      &do
        editd 40
        &pushpoint 2 [ value t%i%xy ]
        split
        &sv sign = 1

```

```

        &end
    &end

&if %sign% = 1 &then &goto split_coord

/* find corner
&sv judge_node = 0
&sv circle_r 80
&sv prx = [ value t%i%x ] - %circle_r%
sel lpoly# = 1 or rpoly# = 1 or lpoly# = 0 or rpoly# = 0
resel lpoly# ne 1 or rpoly# ne 1

&pushpoint 2 [ value t%i%xy ]
&pushpoint 1 %prx% , [ value t%i%y ]
resel circle passthru

&sv arc_number = [show number select]

/* if only one arc is selected, we need to split arc to find a node
&if %arc_number% = 1 &then
    &goto start_split

&sv arc_node_number = %arc_number% + 1

/* Do loop to get the coords of both nodes on all the arcs of the
/* selected arcs

&do j = 1 &to %arc_number% &by 1
    /* Get record number of jth arc.
    &sv arc_id = [show select %j%]
    /* Get coords of nodes of jth arc.
    &sv arc_xy = [show arc %arc_id% nodes]

    /* Set values of coords of nodes.
    &sv fnx%j% = [extract 1 %arc_xy%]
    &sv fny%j% = [extract 2 %arc_xy%]
    &sv fn%j% = [show arc %arc_id% FNODE# ]
    &sv a_id%j% = %arc_id%

    &sv tn%j% = [extract 3 %arc_xy%]
    &sv tny%j% = [extract 4 %arc_xy%]
    &sv tn%j% = [show arc %arc_id% TNODE# ]
&end

/* order arc node from one end to another end
/* find start node
&sv start_find = N
&do j = 1 &to %arc_number% &by 1 &while %start_find% = N

```

```

/* find from FNODE side
&sv judge = 0
&do k = 1 &to %arc_number% &by 1 &while %judge% = 0
  &if [ value fn%j% ] = [ value tn%k% ] &then
    &sv judge = 1
&end

/* check from FNODE side
&do k = 1 &to %arc_number% &by 1 &while %judge% = 0
  &if %k% ne %j% and [ value fn%j% ] = [ value fn%k% ] &then
    &sv judge = 1
&end

&if %judge% = 0 &then
  &do
    &sv start_n = [ value fn%j% ]
    &sv start_find = F
  &end
&end /* end for ( &do j = 1 &to %arc_number% &by 1 )

&if %start_find% = F &then &goto start_done

/* find start node from TNODE side
&do j = 1 &to %arc_number% &by 1 &while %start_find% = N

  /* find from TNODE side
  &sv judge = 0
  &do k = 1 &to %arc_number% &by 1 &while %judge% = 0
    &if [ value tn%j% ] = [ value fn%k% ] &then
      &sv judge = 1
  &end

  /* check from FNODE side
  &do k = 1 &to %arc_number% &by 1 &while %judge% = 0
    &if %k% ne %j% and [ value tn%j% ] = [ value tn%k% ] &then
      &sv judge = 1
  &end

  &if %judge% = 0 &then
    &do
      &sv start_n = [ value tn%j% ]
      &sv start_find = T
    &end
  &end

&end /* end for ( &do j = 1 &to %arc_number% &by 1 )

&label start_done

&sv back_n = 0
/* start to get ordered node coordinates

```

```

&do j = 1 &to %arc_number% &by 1
  &sv find_n = N
  &do k = 1 &to %arc_number% &by 1 &while %find_n% = N
    &if [ value fn%k% ] = [ value start_n ] and ~
      [ value tn%k% ] ne [ value back_n ] &then
      &do
        /* Get coords of nodes of jth arc.
        &sv arc_xy = [ show arc [ value a_id%k% ] nodes ]

        /* Set values of coords of ordered nodes.
        &sv onx%j% = [extract 1 %arc_xy%]
        &sv ony%j% = [extract 2 %arc_xy%]
        &sv on%j% = [show arc [ value a_id%k% ] FNODE# ]
        &sv start_n = [show arc [ value a_id%k% ] TNODE# ]
        &sv back_n = [show arc [ value a_id%k% ] FNODE# ]

        /* get last node coordinates
        &sv ti = %j% + 1
        &sv onx%ti% = [extract 3 %arc_xy%]
        &sv ony%ti% = [extract 4 %arc_xy%]
        &sv on%ti% = [show arc [ value a_id%k% ] TNODE# ]
        &sv find_n = F
      &end /* end if

    &if [ value tn%k% ] = [ value start_n ] and ~
      [ value fn%k% ] ne [ value back_n ] &then
      &do
        /* Get coords of nodes of jth arc.
        &sv arc_xy = [ show arc [ value a_id%k% ] nodes ]

        /* Set values of coords of ordered nodes.
        &sv onx%j% = [extract 3 %arc_xy%]
        &sv ony%j% = [extract 4 %arc_xy%]
        &sv on%j% = [show arc [ value a_id%k% ] TNODE# ]
        &sv start_n = [show arc [ value a_id%k% ] FNODE# ]
        &sv back_n = [show arc [ value a_id%k% ] TNODE# ]

        /* get last node coordinates
        &sv ti = %j% + 1
        &sv onx%ti% = [extract 1 %arc_xy%]
        &sv ony%ti% = [extract 2 %arc_xy%]
        &sv on%ti% = [show arc [ value a_id%k% ] FNODE# ]
        &sv find_n = F
      &end /* end if

    &end /* end of ( do k = 1 )

  &end /* for ( &do j = 1 &to %arc_number% &by 1 )

/* calculate angle to find corner node

```

```

&do j = 3 &to %arc_node_number% &by 1
  /* compute backsight, angle, and to point id & number
  &sv ib = %j% - 2
  &sv ia = %j% - 1
  &sv xb = [ value onx%i% ]
  &sv yb = [ value ony%i% ]
  &sv xa = [ value onx%ia% ]
  &sv ya = [ value ony%ia% ]
  &sv xf = [ value onx%j% ]
  &sv yf = [ value ony%j% ]
  &sv temp_nodex%i% = [ value onx%ia% ]
  &sv temp_nodey%i% = [ value ony%ia% ]
  &sv temp_node%i% = [ value on%i% ]

  /* inverse angle from three points
  invers angle %xb% %yb% %xa% %ya% %xf% %yf%
  &sv angle%i% = [ show inverse angle ]
  &if [ value angle%i% ] > 3.14 &then
    &sv angle%i% = [ calc 6.28 - [ value angle%i% ] ]
&end /* end ( &do j = 3 )

/* find ith true corner coordinates and IDs
&sv min_angle = %angle1%
&sv tic_x%i% = %temp_nodex1%
&sv tic_y%i% = %temp_nodey1%
&sv tic_%i% = %temp_node1%

&sv jj = [ calc %arc_node_number% - 2 ]
&do j = 2 &to %jj% &by 1
  &if [ value angle%j% ] < %min_angle% &then &do
    &sv min_angle = [ value angle%j% ]
    &sv tic_x%i% = [ value temp_nodex%j% ]
    &sv tic_y%i% = [ value temp_nodey%j% ]
    &sv tic_%i% = [ value temp_node%j% ]
  &end /* end if
&end

&if %min_angle% > 1.65 &then &goto start_split

&sv xx = [ value tic_x%i% ]
&sv yy = [ value tic_y%i% ]
&goto node_done

&label start_split
/* if we can't find corner then split arc to get node coordinates
&sv sign = 0
editd 70
sel lpoly# = 1 or rpoly# = 1 or rpoly# = 0 or lpoly# = 0
resel lpoly# ne 1 or rpoly# ne 1

```

```

&pushpoint 1 [ value t%i%xy ]
resel
&pushpoint 1 [ value t%i%xy ]
split
&label split_coord

/* find split node coordinates
&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &sv sp_id = [show select 1 ]
    &sv temp_xy = [ show arc %sp_id% node ]
    &sv temp_xy1 = [extract 1 %temp_xy% ] [extract 2 %temp_xy% ]
    &sv temp_xy2 = [extract 3 %temp_xy% ] [extract 4 %temp_xy% ]
    &sv d1 [ invdistance [ value t%i%xy ] %temp_xy1% ]
    &sv d2 [ invdistance [ value t%i%xy ] %temp_xy2% ]

    &if %d1% < %d2% &then
      &do
        &sv tic_x%i% = [ extract 1 %temp_xy% ]
        &sv tic_y%i% = [ extract 2 %temp_xy% ]
        &sv tic_%i% = [ show arc 1 FNODE# ]
      &end
    &else
      &do
        &sv tic_x%i% = [ extract 3 %temp_xy% ]
        &sv tic_y%i% = [ extract 4 %temp_xy% ]
        &sv tic_%i% = [ show arc 1 TNODE# ]
      &end
    &end
  &end
&else
  &do
    &do j = 1 &to 2 &by 1
      &sv sp_id%j% = [show select %j% ]
      &sv spf%j% = [ show arc [ value sp_id%j% ] FNODE# ]
      &sv spt%j% = [ show arc [ value sp_id%j% ] TNODE# ]
      &sv sp_xy = [show arc [ value sp_id%j% ] nodes]
      &sv spfx%j% = [extract 1 %sp_xy%]
      &sv spfy%j% = [extract 2 %sp_xy%]
      &sv sptx%j% = [extract 3 %sp_xy%]
      &sv spty%j% = [extract 4 %sp_xy%]
    &end

    &if %spf2% = %spt1% &then
      &do
        &sv spx = %spfx2%
        &sv spy = %spfy2%
        &sv sp_node = %spf2%
      &end
    &else

```



```

&do
  &sv spx = %spfx1%
  &sv spy = %spfy1%
  &sv sp_node = %spf1%
&end

&sv tic_x%i% = %spx%
&sv tic_y%i% = %spy%
&sv tic_%i% = %sp_node%

&end

&label node_done

&end

/* write all arc nodes to arc_coordi.dat for late link
/* open a file arc_coordi.dat for store arc coordinates
&if %sc3% ne 0 &then
  &sv .sc3 := [close %sf3%]
&sv .sf3 := [ open arc_coordi.dat .so3 -w ]
&if %so3% ne 0 &then
  &return &inform Could not open arc_coordi.dat file.

&sv total_arc = 0

ef arc
sel lpoly# = 1 or rpoly# = 1 or lpoly# = 0 or rpoly# = 0
resel lpoly# ne 1 or rpoly# ne 1

&sv arc_number = [show number select]

&do i = 1 &to %arc_number% &by 1
  &sv sp_id = [show select %i% ]
  &sv temp_xy = [ show arc %sp_id% node ]
  &sv temp_x1 = [extract 1 %temp_xy% ]
  &sv temp_y1 = [extract 2 %temp_xy% ]
  &sv temp_x2 = [extract 3 %temp_xy% ]
  &sv temp_y2 = [extract 4 %temp_xy% ]

  /* find if the point exist and is not tic point
  &sv judge = 0
  &do j = 1 &to %total_arc% &by 1
    &sv dx = [ abs [ calc %temp_x1% - [ value cnodex%j% ] ] ]
    &sv dy = [ abs [ calc %temp_y1% - [ value cnodey%j% ] ] ]
    &if %dx% < 1 and %dy% < 1 &then
      &sv judge = %judge% + 1
  &end

```

```

&do j = 1 &to %tic_number% &by 1
  &sv dx = [ abs [ calc %temp_x1% - [ value tic_x%j% ] ] ]
  &sv dy = [ abs [ calc %temp_y1% - [ value tic_y%j% ] ] ]
  &if %dx% < 0.1 and %dy% < 1.1 &then
    &sv judge = %judge% + 1
  &end

&if %judge% = 0 &then
  &do
    &sv total_arc = %total_arc% + 1
    &sv cnodex%total_arc% = %temp_x1%
    &sv cnodey%total_arc% = %temp_y1%
    &s .sw3 := [write %.sf3% %temp_x1%]
    &s .sw3 := [write %.sf3% %temp_y1%]
  &end

&sv judge = 0
&do j = 1 &to %total_arc% &by 1
  &sv dx = [ abs [ calc %temp_x2% - [ value cnodex%j% ] ] ]
  &sv dy = [ abs [ calc %temp_y2% - [ value cnodey%j% ] ] ]
  &if %dx% < 1 and %dy% < 1 &then
    &sv judge = %judge% + 1
  &end

&do j = 1 &to %tic_number% &by 1
  &sv dx = [ abs [ calc %temp_x2% - [ value tic_x%j% ] ] ]
  &sv dy = [ abs [ calc %temp_y2% - [ value tic_y%j% ] ] ]
  &if %dx% < 0.1 and %dy% < 0.1 &then
    &sv judge = %judge% + 1
  &end

&if %judge% = 0 &then
  &do
    &sv total_arc = %total_arc% + 1
    &sv cnodex%total_arc% = %temp_x2%
    &sv cnodey%total_arc% = %temp_y2%
    &s .sw3 := [write %.sf3% %temp_x2%]
    &s .sw3 := [write %.sf3% %temp_y2%]
  &end

&end /* end for find arc_node coordinates
&s .sc3 := [close %.sf3%]

/* start to split coverage boundary recording pls nodes
/* =====
/* find pls boundary node coordinates
&if %sc1% ne 0 &then
  &sv .sc1 = [ close %.sf1% ]

```

```

&sv .sf1 := [ open bound_xy.dat .so1 -r ]
&if %.so1% ne 0 &then
  &return &inform Could not open bound_xy.dat file.

&sv bound_num = 0
&s record_num := [read %.sf1% .sr1]
&do i = 1 &to %record_num% &by 1
  &s record1 := [read %.sf1% .sr1]
  &s record2 := [read %.sf1% .sr1]

  &sv jadge = 0
  &do j = 1 &to %tic_number%
    &sv dx = [ abs [ calc %record1% - [ value t%j%x ] ] ]
    &sv dy = [ abs [ calc %record2% - [ value t%j%y ] ] ]

    &if %dx% < 0.1 and %dy% < 0.1 &then
      &sv jadge = %jadge% + 1
    &end

  &do j = 1 &to %bound_num%
    &sv dx = [ abs [ calc %record1% - [ value boundx%j% ] ] ]
    &sv dy = [ abs [ calc %record2% - [ value boundy%j% ] ] ]

    &if %dx% < 1 and %dy% < 1 &then
      &sv jadge = %jadge% + 1
    &end

  &if %jadge% = 0 &then
    &do
      &sv bound_num = %bound_num% + 1
      &sv boundx%bound_num% = %record1%
      &sv boundy%bound_num% = %record2%
    &end
  &end
&s .sc1 := [close %.sf1%]

/* open a file node_coord.dat for store node coordinates
&if %.sc2% ne 0 &then
  &sv .sc2 := [close %.sf2%]
&sv .sf2 := [ open node_coord.dat .so2 -w ]
&if %.so2% ne 0 &then
  &return &inform Could not open node_coord.dat file.

&sv total_node = 0

ef arc
editd 100
&sv d_unit = 2

```

```

&do i = 1 &to %bound_num% &by 1
  sel lpoly# = 1 or rpoly# = 1 or rpoly# = 0 or lpoly# = 0
  resel lpoly# ne 1 or rpoly# ne 1

  &sv xb [ value boundx%i% ]
  &sv yb [ value boundy%i% ]

  &pushpoint 1 %xb% %yb%
  resel
  &pushpoint 1 %xb% %yb%
  split

  /* find split node coordinates
  &sv arc_number = [show number select]

  &if %arc_number% = 1 &then
    &do
      &sv sp_id = [show select 1 ]
      &sv temp_xy = [ show arc %sp_id% node ]
      &sv temp_xy1 = [extract 1 %temp_xy% ] [extract 2 %temp_xy% ]
      &sv temp_xy2 = [extract 3 %temp_xy% ] [extract 4 %temp_xy% ]
      &sv d1 [ invdistance %xb% %yb% %temp_xy1% ]
      &sv d2 [ invdistance %xb% %yb% %temp_xy2% ]

      &if %d1% < %d2% &then
        &do
          &sv px = [ extract 1 %temp_xy% ]
          &sv py = [ extract 2 %temp_xy% ]
        &end
      &else
        &do
          &sv px = [ extract 3 %temp_xy% ]
          &sv py = [ extract 4 %temp_xy% ]
        &end

      &sv arc_number = [show number select]
      &do j = 1 &to 5 &while %arc_number% = 1
        &sv d_unit = [ calc %j% * %d_unit% ]
        &sv angle = [ show arc %sp_id% orientation %px% %py% ]
        &sv dx = [ calc %d_unit% * [ cos %angle% ] ]
        &sv dy = [ calc %d_unit% * [ sin %angle% ] ]

        &sv arc_number = [show number select]
        &if %arc_number% = 1 &then
          &do
            &pushpoint 1 [ calc %px% + %dx% ] [ calc %py% + %dy% ]
            split
          &end
        &end
      &end
    &end
  &end

```

```

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &pushpoint 1 [ calc %px% + %dx% ] [ calc %py% - %dy% ]
    split
  &end

```

```

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &pushpoint 1 [ calc %px% - %dx% ] [ calc %py% - %dy% ]
    split
  &end

```

```

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &pushpoint 1 [ calc %px% - %dx% ] [ calc %py% + %dy% ]
    split
  &end

```

```

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &pushpoint 1 [ calc %px% + %d_unit% ] [ calc %py% ]
    split
  &end

```

```

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &pushpoint 1 [ calc %px% - %d_unit% ] [ calc %py% ]
    split
  &end

```

```

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &pushpoint 1 [ calc %px% ] [ calc %py% - %d_unit% ]
    split
  &end

```

```

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &pushpoint 1 [ calc %px% ] [ calc %py% + %d_unit% ]
    split
  &end

```

```

&end /* end for j = 1

&end /* end for &do arc_num = 1

&sv arc_number = [show number select]
&if %arc_number% = 1 &then
  &do
    &s .sw2 := [write %.sf2% %px%]
    &s .sw2 := [write %.sf2% %py%]
    &s .sw2 := [write %.sf2% %xb%]
    &s .sw2 := [write %.sf2% %yb%]
    &sv total_node = %total_node% + 1
  &end
&else
  &do
    &do j = 1 &to 2 &by 1
      &sv sp_id%j% = [show select %j% ]
      &sv spf%j% = [ show arc [ value sp_id%j% ] FNODE# ]
      &sv spt%j% = [ show arc [ value sp_id%j% ] TNODE# ]
      &sv sp_xy = [show arc [ value sp_id%j% ] nodes]
      &sv spfx%j% = [extract 1 %sp_xy%]
      &sv spfy%j% = [extract 2 %sp_xy%]
      &sv sptx%j% = [extract 3 %sp_xy%]
      &sv spty%j% = [extract 4 %sp_xy%]
    &end

    &if %spf2% = %spt1% &then
      &do
        &sv spx = %spfx2%
        &sv spy = %spfy2%
        &sv sp_node = %spf2%
      &end
    &else
      &do
        &sv spx = %spfx1%
        &sv spy = %spfy1%
        &sv sp_node = %spf1%
      &end

    &s .sw2 := [write %.sf2% %spx%]
    &s .sw2 := [write %.sf2% %spy%]
    &s .sw2 := [write %.sf2% %xb%]
    &s .sw2 := [write %.sf2% %yb%]
    &sv total_node = %total_node% + 1

  &end

&end /* end for ( &do i = 1 &to %bound_num% &by 1 )

```

```

&sv .sc2 := [close %.sf2%]

/* links arc to pls, & node to node
/* =====
/* Select all the arcs of the external polygon and generalize them.
sel lpoly# = 1 or rpoly# = 1 or rpoly# = 0 or lpoly# = 0
resel lpoly# ne 1 or rpoly# ne 1

generalize 1000

/* Set appropriate edit data for edgematching and perform the edgematching
editf link
/* Set snapcover to section line coverage in appropriate directory.
/* snapcov %.PATH%/%.COUNTY%
snapcov %snap_pls%

/* link node to node
/* -----
snapfeature node node
snapping closest 80
linkfeature node node

/* Read the coordinates of the nodes of the external polygon and use them to
/* link to the section lines.

/* link tics
&do i = 1 &to %tic_number% &by 1
  &pushpoint 2 [ value tic_x%i% ] [ value tic_y%i% ]
  &pushpoint 2 [ value t%i%xy ]
&end

/* link boundary node
/* read node from boundary nodes node_coord.dat
&if %.sc4% ne 0 &then
  &sv .sc4 := [close %.sf4%]
&sv .sf4 := [open node_coord.dat .so4 -r]
&if %.so4% ne 0 &then
  &return &inform Could not open file node_coord.dat.

&do i = 1 &to %total_node% &by 1
  &sv record1 := [read %.sf4% .sr4 ]
  &sv record2 := [read %.sf4% .sr4 ]
  &sv record3 := [read %.sf4% .sr4 ]
  &sv record4 := [read %.sf4% .sr4 ]
  &pushpoint 2 %record1% %record2%
  &pushpoint 2 %record3% %record4%
&end
&sv .sc := [ close %.sf4% ]

```

```

&pushpoint 9 0 0

/* Add links using the node coords in pushpoint.
add

/* link node to arc
snapfeature node arc
snapping closest 80
linkfeature node arc

/* Read the coordinates of the nodes of the external polygon and use them to
/* link to the section lines.

&if %.sc5% ne 0 &then
    &sv .sc5 := [close %.sf5%]

&s .sf5 := [open arc_coord.dat .so5 -read ]
&if %.so5% ne 0 &then
    &return &inform Could not open file arc_coord.dat.

&do i = 1 &to %total_arc% &by 1 &while %.sr5% ne 102
    &s record1 := [read %.sf5% .sr5 ]
    &s record2 := [read %.sf5% .sr5 ]
    &pushpoint 2 %record1% %record2%
    &pushpoint 2 %record1% %record2%

&end

&s .sc5 := [close %.sf5%]
&pushpoint 9 0 0

/* Add links using the node coords in pushpoint.
add
adjust

/* Delete the line which forms a box around the section.
&if %d_type% = d &then
    &do
        editfeature arcs
        select ATTRIBUTE = 1
        delete
    &end

save
&return

```

3. Final Procedures AML

```

/*=====
/* This aml works fro soils data final pcedures:

```



```

/* They include:
/*   - append drains, water, spots coverages to
/*     county size
/*   - rename coverage
/*   - Jion table
/*   - replace tic points with county coverage tics
/*   - idedit, index
/*   - frequency soils
/*       created by Ruifeng Xi
/*       ISU GIS lab. Apr.15, 1998
/*=====

/* set county neme, number, and path
/* -----
&sv .county = page
&sv .county_num = 73
&sv .work_dir = /home/gis15/soils/%.county%
&sv .aml_dir = /home/gis13/soils/aml

&severity &error &ignore
&severity &warning &ignore

/* copy *.dbf file to work directory
&sys cp /home/gis13/soils/ispaid/isp%.county_num%.dbf %.work_dir%/dnr
&sys cp /home/gis13/soils/ispaid/drains.dbf %.work_dir%/dnr
&sys cp /home/gis13/soils/ispaid/symbols.dbf %.work_dir%/dnr

/* copy all soils coverage to dnr directory
ws coe/soils

&s township_num := [filelist t*r* township.dat -cover]
&s fileunit1 := [open township.dat openstat1 -read]

&do i = 1 &to %township_num% &by 1
    &s township_name = [read %fileunit1% readstat1]

    &if [ exists %.work_dir%/dnr/%township_name% -cover ] &then
        kill %.work_dir%/dnr/%township_name% all
        copy %township_name% %.work_dir%/dnr/%township_name%
&end

&s closestat1 := [close %fileunit1%]
ws %.work_dir%

/* append all drains coverage to county size and copy it to dnr directory
ws coe/drains
&s tf5 := [open appends.aml to5 -w]
&if %to5% ne 0 &then &return &inform Could not open append list

```

```
&s record = [quote append drain_%.county_num% line all]
&s tw5 := [write %tf5% %record%]
```

```
&s township_num := [filelist t*r* township.dat -cover]
&s fileunit1 := [open township.dat openstat1 -read]
```

```
&do i = 1 &to %township_num% &by 1
    &s township_name = [read %fileunit1% readstat1]
    &s tw5 := [write %tf5% %township_name%]
&end
```

```
&s tw5 := [write %tf5% end]
&s closestat1 := [close %fileunit1%]
&s closestat2 := [close %tf5%]
```

```
&if [ exists drain_%.county_num% -cover ] &then
    kill drain_%.county_num% all
```

&run appends.aml

```
clean drain_%.county_num% # 1 5 line
dissolve drain_%.county_num% drain_%.county_num%d attribute line
kill drain_%.county_num% all
rename drain_%.county_num%d drain_%.county_num%
build drain_%.county_num% line
```

```
copy drain_%.county_num% %.work_dir%/dnr/drain_%.county_num%
ws %.work_dir%
```

/* append all spots coverage county size and copy it to dnr directory

```
ws coe/spots
&s tf5 := [open appends.aml to5 -w]
&if %to5% ne 0 &then &return &inform Could not open append list
```

```
&s record = [quote append spot_%.county_num% point all]
&s tw5 := [write %tf5% %record%]
```

```
&s township_num := [filelist t*r* township.dat -cover]
&s fileunit1 := [open township.dat openstat1 -read]
```

```
&do i = 1 &to %township_num% &by 1
    &s township_name = [read %fileunit1% readstat1]
    &s tw5 := [write %tf5% %township_name%]
&end
```

```
&s tw5 := [write %tf5% end]
```

```
&s closestat1 := [close %fileunit1%]
&s closestat2 := [close %tf5%]
&s closestat := [close -all]
```

```

&if [ exists spot_%.county_num% -cover ] &then
    kill spot_%.county_num% all

&run appends.aml
copy spot_%.county_num% %.work_dir%/dnr/spot_%.county_num%
ws %.work_dir%

/* append all water coverage to county size and copy it to dnr directory
ws coe/water

&s tf5 := [open appends.aml to5 -w]
&if %to5% ne 0 &then &return &inform Could not open append list

&s record = [quote append water_%.county_num% poly all]
&s tw5 := [write %tf5% %record%]
&s township_num := [filelist t*r* township.dat -cover]
&s fileunit1 := [open township.dat openstat1 -read]

&do i = 1 &to %township_num% &by 1
    &s township_name = [read %fileunit1% readstat1]
    &s tw5 := [write %tf5% %township_name%]
&end
&s tw5 := [write %tf5% end]

&s closestat1 := [close %fileunit1%]
&s closestat2 := [close %tf5%]
&s closestat := [close -all]

&if [ exists water_%.county_num% -cover ] &then
    kill water_%.county_num% all

&if %township_num% = 0 &then &goto next
&run appends.aml

/* delete pseudo node in water coverage
ae
edit %township_name%
ef arc
sel all
unsplit none
save
q

clean water_%.county_num% # 2 2

dissolve water_%.county_num% water_%.county_num%w attribute poly
kill water_%.county_num% all
rename water_%.county_num%w water_%.county_num%
build water_%.county_num% line
build water_%.county_num%

```

```

copy water_%.county_num% %.work_dir%/dnr/water_%.county_num%

&label next

/* join soils coverage with ips** table
/* join drains, spots coverages with drains.dbf and symbol.dbf
/* -----
ws %.work_dir%/dnr
&s township_num := [filelist t*r*s township.dat -cover]
&s fileunit1 := [open township.dat openstat1 -read]
tables
&do i = 1 &to %township_num% &by 1
    &s township_name = [read %fileunit1% readstat1]
    ADDITEM %township_name%.pat SMS 5 5 c
    sel %township_name%.pat
    CALCULATE SMS = label
&end
q
&s closestat := [close -all]

/* covert dbf file to info file
&if [ exists isp%.county_num% -info ] &then &goto next1
dbaseinfo isp%.county_num%.dbf isp%.county_num%

&label next1
&if [ exists drains -info ] &then &goto next2
dbaseinfo drains.dbf drains

&label next2
&if [ exists symbols -info ] &then &goto next3
dbaseinfo symbols.dbf symbols

&label next3
&s township_num := [filelist t*r*s township.dat -cover]
&s fileunit1 := [open township.dat openstat1 -read]

&do i = 1 &to %township_num% &by 1
    &s township_name = [read %fileunit1% readstat1]
    &sv t = [ substr %township_name% 2 2 ]
    &sv r = [ substr %township_name% 5 2 ]
    JOINITEM %township_name%.pat isp%.county_num%
    %township_name%.pat SMS SMS
    rename %township_name% s%t%n%r%w
&end

&s closestat1 := [close %fileunit1%]

JOINITEM drain_%.county_num%.aat drains drain_%.county_num%.aat attribute
attribute

```

JOINITEM spot_%.county_num%.pat symbols spot_%.county_num%.pat attribute
attribute

/* replace tic points with county tics

/* -----
&sv cover_num := [filelist * cover.dat -cover]
&if %cover_num% <= 0 &then &return &inform Error in generating list of cover.dat

&s fileunit1 := [open cover.dat openstat1 -read]
&do i = 1 &to %cover_num% &by 1
 &sv cover_name = [read %fileunit1% readstat1]
 ae
 edit %cover_name%
 mape %cover_name%
 de all
 draw
 ef tic
 sel all
 delete
 get %.work_dir%/%.county%_pls
 save
 quit
&end

/* soils coverage for final processing

/* -----
&s soil_num := [filelist s*w soil.dat -cover]

&if %soil_num% <= 0 &then
 &return &inform Error in generating list of soil coverages

&s tf1 := [open soil.dat to1 -read]
&if %to1% ne 0 &then &return &inform Could not open list of soil

&do i = 1 &to %soil_num% &by 1
 &s soil_name = [read %tf1% tr1]
 build %soil_name% line
 tables
 sel %soil_name%.aat
 calc %soil_name%-id = %soil_name%#
 quit
 idedit %soil_name% line
 index %soil_name% all
 frequency %soil_name%.pat %soil_name%.freq
 label
 end
 area
 end
&end

```

/* final steps for drains, water, spots coverage
/*-----
tables
    sel drain_%.county_num%.aat
    calc drain_%.county_num%-id = drain_%.county_num%#
quit

idedit drain_%.county_num% line
index drain_%.county_num% all

tables
    sel spot_%.county_num%.pat
    calc spot_%.county_num%-id = spot_%.county_num%#
quit

idedit spot_%.county_num% point
index spot_%.county_num% all

build water_%.county_num% line
tables
    sel water_%.county_num%.pat
    calc water_%.county_num%-id = water_%.county_num%#
quit

idedit water_%.county_num% line
index water_%.county_num% all

/* export coverage to E00 file
/*-----
&run %.aml_dir%/export
&if [ delete township.dat ] = 0 &then
    &type township.dat deleted successfully

&if [ delete cover.dat ] = 0 &then
    &type township.dat deleted successfully

&sys rm *.dbf
&return

```

REFERENCE LIST

1. *Global Positioning System Standard Positioning Service Specification*, 2nd Edition, June 2, 1995. Available on line from <http://www.navcen.uscg.gov/pubs/gps/sigspec/default.htm>.
2. *GPS Interface Control Document ICD-GPS-200C*, NAVSTAR GPS Space Segment and Navigation User Interfaces, Nov. 27, 2000.
3. Hoffmann-Wellenhof, B. H. Lichtenegger, and J. Collins. 1994. *GPS: Theory and Practice*. 3rd edit. New York: Springer-Verlag.
4. Institute of Navigation. 1980, 1984, 1986, 1993. *Global Positioning System Monographs*. Washington, DC: The Institute of Navigation.
5. Kaplan, Elliott D. ed. 1996. *Understanding GPS: Principles and Applications*. Boston: Artech House Publishers.
6. Leick, Alfred. 1995. *GPS Satellite Surveying*. 2nd. Edition. New York: John Wiley & Sons.
7. Parkinson, Bradford W. and James J. Spilker. eds. 1996. *Global Positioning System: Theory and Practice*. Volumes I and II.

- Washington, DC: American Institute of Aeronautics and Astronautics, Inc.
8. Wells, David, ed. 1989. *Guide to GPS positioning*. Fredericton, NB, Canada: Canadian GPS Associates.
 9. Indorante, S.J., R.L.McLeese, R.D.Hammer, B.W.Thompson, D.L.Alexander. 1996. *Positioning soil survey for the 21st century*. J. Soil and Water Conservation 51:21-28.
 10. Valentine, K.W.G., W.C. Naughton and M. Nava. 1981. A *questionnaire to users of soil maps in British Columbia: Results and implications for design and content*. Can. J. Soil Sci. 61: 123-135.
 11. GIS by ESRI, Arc/Info version 7, *AML Commands*, Environmental Systems Research Institute, INC. 1994.
 12. Donna J. Peuquet, *Geographic Information Systems*, London , New York, Philadelphia, 1990.
 13. Indorante, S.J., R.L.McLeese, R.D.Hammer, B.W.Thompson, D.L.Alexander. 1996. *Positioning soil survey for the 21st century*. J. Soil and Water Conservation 51:21-28.
 14. Valentine, K.W.G., W.C. Naughton and M. Nava. 1981. A *questionnaire to users of soil maps in British Columbia: Results and implications for design and content*. Can. J. Soil Sci. 61: 123-135.

15. GIS by ESRI, Arc/Info version 7, *AML Commands*, Environmental Systems Research Institute, INC. 1994.
16. Donna J. Peuquet, *Geographic Information Systems*, London, New York, Philadelphia, 1990.
17. OMNI User Manual, Geodetic Research and Development Laboratory, National Geodetic Survey (NGS), 1995.
18. Surveying, ninth edition, Francis H. Moffitt, Harry Bouchard, 106p, New York, NY, 1992.
19. Global Positioning System, theory and practice, Hofmann-Wellenhof, B.; New York, Springer-Verlag, 1997.
20. GPS: a case study in generality and problem solving, Ernst, George W., New York, Academic Press, 1969.
21. GPS for GIS data collection, Fort Belvoir, U.S. Army Topographic Engineering Center, 1997.
22. GPS for land surveyors, Van Sickle, Jan., Chelsea, Mich; Ann Arbor Press, 1996.
23. Global Positioning System: a guide for the approval of GPS receiver installation and operation, Wright, M., Washington, DC., 1996.
24. The global positioning system: a shared national asset : recommendations for technical improvements and enhancements,

- National Research Council (U.S.). , Committee on the Future of the Global Positioning System, Washington, D.C. National Academy Press, 1995.
25. The Global Positioning System and GIS: an introduction. Kennedy, Michael, Chelsea, MI, Ann Arbor Press, Inc., 1996.
 26. Global Positioning System information retrieval methodology, Synstelien, Larry Duane, 1989.
 27. Global Positioning System Reobservations Over the Eastern United States Strain Monitoring Network, NUREG/CR-6473, U.S. Nuclear Regulatory Commission, 1995.
 28. Global positioning system: theory and application, Washington, DC. American Institute of Aeronautics and Astronautics, 1996.
 29. Global Positioning Systems technology and its application in environmental programs, Las Vegas, NV. Environmental Monitoring Systems Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, 1992.
 30. GPS for land surveyors, Van Sickle, Jan. Chelsea, Mich., Ann Arbor Press, 1996.
 31. Understanding GPS: principles and applications, Boston, Artech House, 1996.

32. Error modeling for differential GPS <microform>, Bierman, Gregory S., Cambridge, Mass., Charles Stark Draper Laboratory Inc., Washington, DC, National Aeronautics and Space Administration; Springfield, Va., National Technical Information Service, distributor, 1995.
33. Understanding the Navstar: GPS, GIS, and IVHS, Logsdon, Tom, New York, Van Nostrand Reinhold, 1995.
34. Evaluation of GPS position and attitude determination for automated rendezvous and docking missions <microform>, DiPrinzio, Marc David, Hampton, National Aeronautics and Space Administration, Langley Research Center, 1994.
35. Toward 10 GPS geodesy <microform>, vector baselines, earth rotation and reference frames, Washington, DC. National Aeronautics and Space Administration, National Technical Information Service, distributor, 1994.
36. Precision GPS ephemerides and baselines <microform>, final report for NASA grant no. NAGW-2717, January 1991-June 1992. Austin, Tex., Center for Space Research, University of Texas at Austin; Washington, D.C., National Aeronautics and Space Administration, 1992.

37. Systems analysis for a navigation receiver allowing simultaneous utilization of the GLONASS and NAVSTAR GPS satellite navigation systems, Popeck, Charles Alden, 1992.
38. Error analysis of real time and post processed orbit determination of GFO using GPS tracking <microform>, Schreiner, William S., Washington, DC., National Aeronautics and Space Administration; Springfield, Va., National Technical Information Service, distributor, 1991.
39. Extending the National Geodetic Survey standard GPS orbit formulas, Remondi, Benjamin W., Rockville, MD, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration, National Ocean Service, Charting and Geodetic Services, National Geodetic Information Branch, 1989.
40. Orthometric height difference recovery tests from GPS observations and gravimetry, Erck, E. Stephen, 1989.
41. Self-contained GPS integrity monitoring using a censored Kalman filter, McBurney, Paul William, 1988.
42. Effect of process noise covariance selection on GPS receiver Kalman filter stability, Wissink, Kevin, 1987.
43. Guide to GPS positioning, Fredericton, New Brunswick, Canadian GPS Associates, 1987.

44. Analysis of a GPS aided inertial navigation system using the delayed state Kalman filter, McBurney, Paul William, 1986.
45. Positioning with GPS-1985: proceedings, First International Symposium on Precise Positioning with the Global Positioning System, Rockville, Maryland, April 15-19, 1985. International Symposium on Precise Positioning with the Global Positioning System. Rockville, MD. : U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration, National Ocean Service, National Geodetic Information Center, NOAA, 1985.
46. Application of GPS to geodesy: a combination problem in estimation and large-scale hypothesis testing, Hwang, Patrick Y. C. 1983.
47. GPS for GIS data collection, U.S. Army Topographic Engineering Center, 1997.
48. Descriptions, photographs, and coordinates for Global Positioning System stations at Aniakchak Crater, Alaska <microform>, Yamashita, Kenneth M. U.S. Dept. of the Interior, U.S. Geological Survey, 1996.
49. The Global Positioning System and GIS: an introduction, Kennedy, Michael, Chelsea, MI : Ann Arbor Press, Inc., 1996.

50. Centimeter-level orthometric heights at reference points along Florida's Big Bend coastline from Global Positioning System (GPS) static surveys <microform>, St. Petersburg, FL, U.S. Dept. of the Interior, U.S. Geological Survey, Center for Coastal Geology, 1995.
51. Civil utilization of Global Positioning System (GPS) <microform>. United States., Federal Aviation Administration. Washington, D.C. U.S. Dept. of Transportation, Federal Aviation Administration, 1995.
52. Analytical estimation of carrier multipath bias on GPS position measurements <microform>, Volk, C. Michael. Boulder, Colo., U.S. Dept. of Commerce, Technology Administration, National Institute of Standards and Technology; Washington, U.S. G.P.O., 1994.
53. Global positioning technology <microform>, opportunities for greater federal agency joint development and use: report to congressional requesters, United States., General Accounting Office. Washington, D.C., 1994.
54. Modeling global positioning system effects in the TLC/NLC model, Allen, Patrick D. Publisher: Santa Monica, CA : Rand, 1994.
55. Current and planned use of the Navstar global positioning system <microform>, Thesis, Harold L. Washington, DC, National

- Aeronautics and Space Administration, Springfield, Va., National Technical Information Service, distributor, 1993
56. Investigation for improving Global Positioning System (GPS) orbits using a discrete sequential estimator and stochastic models of selected physical processes <microform>, Goad, Clyde C., Columbus, Ohio, Ohio State University, Washington, DC, National Aeronautics and Space Administration, Springfield, Va. 1993.
 57. NASA Space Geodesy Program <microform>: catalogue of site information, Bryant, M. A. Greenbelt, Md., National Aeronautics and Space Administration, Goddard Space Flight Center; Springfield, Va., National Technical Information Service, 1993.
 58. Accuracy of Global Positioning System broadcast orbits for relative surveys, Remondi, Benjamin W. Publisher: Rockville, MD, U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration, National Ocean Service, Charting and Geodetic Services, National Geodetic Information Branch, NOAA, 1989.
 59. Global Positioning System information retrieval methodology, Synsteliën, Larry Duane. 1989.
 60. Guide to GPS positioning / Publisher: Fredericton, New Brunswick: Canadian GPS Associates, 1987.

61. Effects of sudden ionospheric disturbances on the Global Positioning System receiver, Martinez, Boris Nestor. 1985.
62. Positioning with GPS-1985: proceedings, First International Symposium on Precise Positioning with the Global Positioning System, Rockville, Maryland, April 15-19, 1985.
63. Interpolation, Steffensen, J. F., Baltimore, The Williams & Wilkins Company, 1927.
64. Interpolation and approximation, Davis, Philip J., New York, Blaisdell Pub. Co. 1963.
65. Interpolation and approximation, Davis, Philip J., New York, Dover Publications, 1975.
66. Interpolation of functions / Author: Szabados, J.; Singapore; Teaneck, NJ, World Scientific, 1990.
67. Interpolation of ground water data using multiquadric equations, Anderson, Timothy John, 1986.
68. Interpolation of linear operators, Krein, S. G.; Providence, R.I., American Mathematical Society, 1982.
69. Interpolation of rational matrix functions, Ball, Joseph A.; Basel, Boston, Birkhauser Verlag, 1990.
70. Interpolation theory and its applications, Sakhnovich, L. A.; Dordrecht, Boston, Kluwer Academic Publishers, 1997.

71. Interpolational and extremal properties of L-spline functions, Morsche, H. G., Helmond, Netherlands, Wibro, 1982.
72. Geometry and interpolation of curves and surfaces, McLeod, Robin J. Y. Cambridge; New York, Cambridge University Press, 1998.
73. Numerical modeling of pulsed eddy current techniques for detection of corrosion and cracks, Dasoju, Kiran Kumar., 1997.
74. One dimensional spline interpolation algorithms, Spath, Helmuth, Wellesley, Mass, A K Peters, 1995.
75. Two dimensional spline interpolation algorithms, Spath, Helmuth, Wellesley, Mass. A.K. Peters, 1995.
76. Error inequalities in polynomial interpolation and their applications, Agarwal, Ravi P., Dordrecht, Netherlands; Boston, Kluwer Academic, 1993.
77. New aspects in interpolation and completion theories, Basel; Boston, Birkhauser Verlag, 1993.
78. Spline functions and multivariate interpolations, Bojanov, B. D.; Dordrecht, Boston, Kluwer Academic Publishers, 1993.
79. Interpolation of functions, Szabados, J., Singapore; Teaneck, NJ, World Scientific, 1990.

80. Surveying, Theory and Practice, Seventh Edition, James M Anderson, WCB McGraw-Hill, 1998
81. Feasibility study, Mandel, Betty A.; Fort Belvoir, VA, U.S. Army Corps of Engineers, Engineer Topographic Laboratories; Gaithersburg, MD, Center for Applied Mathematics, National Bureau of Standards, 1987.
82. Curve and surface fitting: an introduction, Lancaster, Peter, London. Orlando, Academic Press, 1986.
83. Error-free polynomial matrix computations, Krishnamurthy, E. V.; New York, Springer-Verlag, 1985.
84. A survey of curve and surface fitting, Lancaster, Peter, Calgary, Lancaster, 1977.
85. Cardinal spline interpolation, Schoenberg, I. J., Philadelphia, Society for Industrial and Applied Mathematics, 1973.
86. Spline analysis, Schultz, Martin H.; Englewood Cliffs, N.J., Prentice-Hall 1972.
87. Technical papers, 1990 ACSM/ASPRS Annual Convention, March 18-23, 1990, Denver, Colorado, ACSM 50th annual meeting, ASPRS 56th annual meeting. ASPRS-ACSM Convention, Bethesda, MD; American Society for Photogrammetry and Remote

- Sensing and American Congress on Surveying and Mapping,
1990.
88. Surveying, Kahmen, Heribert, Berlin, New York, W. de Gruyter,
1988.
 89. Geodesy, an introduction, Torge, Wolfgang, Berlin; New York,
Gruyter, 1980.
 90. Geodesy, the concepts, Vanicek, Petr, North-Holland Pub. Co.,
N.Y. 1982.
 91. EDM traverses: measurement, computation, and adjustment,
Bird, R. G.; Harlow, Essex, England: Longman Scientific &
Technical; New York, 1989.
 92. Geodetic levelling, Schomaker, M. Christine. Rockville, Md. U.S.
Dept. of Commerce, National Oceanic and Atmospheric
Administration, National Ocean Survey, 1981.
 93. Site surveying and leveling, Clancy, John. London, E. Arnold,
1981.
 94. Advanced engineering mathematics with Mathematic and Matlab,
Malek-Madani, Reza. Addison-Wesley, 1998.
 95. System analysis and signal processing: with emphasis on the use
of MATLAB. Denbigh, Philip. Harlow, England; Addison-Wesley,
1998.

96. Introduction to random signals and applied Kalman filtering: with MATLAB exercises and solutions, Brown, Robert Grover. New York, Wiley, 1997.
97. Numerical methods with MATLAB: a resource for scientists and engineers, Borse, G. J. Boston, PWS Publishing, 1997.
98. Digital filters and signal processing: with MATLAB exercises, Jackson, Leland B., Boston Kluwer Academic Publishers, 1996.
99. The MATLAB handbook, Part-Enander, Eva. Harlow, England, Addison-Wesley, 1996.
100. The use of high resolution height data in the computation of high precision geoid undulations on the Island of Maui, Smith, Dru A., Dept. of Geodetic Science and Surveying, Ohio State University, 1992.
101. On the improvement of gravimetric geoid undulations and oceanographic dynamic topography through a combination with satellite altimetry and the application of a modified Kalman filter algorithm, Dru Alan Smith, Columbus, Ohio, Dept. of Geodetic Science and Surveying, The Ohio State University, 1996.
102. Applications of geodesy to engineering, convened and edited by Klaus Linkwitz, Viktor Eisele, Hans-Joachim Mönicke. New York, Springer-Verlag, 1993.

103. Analytical estimation of carrier multipath bias on GPS position measurements [microform], Michael Volk, Judah Levine, Boulder, Colo. U.S. Dept. of Commerce, Technology Administration, National Institute of Standards and Technology; Washington, DC, 1994.
104. Determination of the geoid, present and future, Symposium no. 106, Milan, Italy, June 11-13, 1990.
105. Ellipsoidal corrections for geoid undulation computations, Richard H. Rapp, Columbus, Ohio, Research Foundation, The Ohio State University, 1981.
106. Geoid and its geophysical interpretations, Petr Vaníček, Nikolaos T. Christou, Boca Raton, FL, CRC Press, 1994.
107. Gravity and geoid: joint symposium of the International Gravity Commission and the International Geoid Commission, Graz, Austria, September 11-17, 1994.
108. The use of high resolution height data in the computation of high precision geoid undulations on the Island of Maui, Dru A. Smith, Columbus, OH, Dept. of Geodetic Science and Surveying, Ohio State University, 1992.

109. Reducing the error of geoid undulation computations by modifying Stokes' function, Christopher Jekeli, Columbus, Ohio: Dept. of Geodetic Science, The Ohio State University, 1980.
110. Splines and variational methods, Prenter, P. M., New York, Wiley, 1975.
111. Quadratic Spline Collocation methods for elliptic Partial Differential Equations, Christina C. Christara. 1990.
112. Least squares collocation applied to local gravimetric solutions from satellite gravity gradiometry data [microform], John W. Robbins. 1985.
113. Possibilities of low-low satellite tracking for local geoid improvement, Jan Krynski. 1978.
114. Least squares collocation applied to local gravimetric solutions from satellite gravity gradiometry data, John Wright Robbins. 1985.
115. Global geoid and gravity anomaly predictions using collocation and point mass techniques, Robert P. Bessette, George Hadgigeorge. 1984.
116. The least squares collocation from the adjustment point of view and related topics, Georges Blaha. 1976.

117. Gurtner, W. and Mader, G.L., "THE RINEX FORMAT: CURRENT STATUS, FUTURE DEVELOPMENTS", Proceedings of the Second International Symposium on Precise Positioning with the Global Positioning System. pp. 977-992. Ottawa, Canada, September 1990.
118. Environmental Systems Research Institute. "Avenue Customization and Application Development for Arcview", 1996.
119. Amir H. Razavi and Valerie Warwick, "ArcView GIS/Avenue Programmer's Reference, Second Edition", 1997.
120. Environmental Systems Research Institute. Understanding GIS - The ARC/INFO Method. Red Island, CA: Environmental Systems Research Institute, 1994.
121. Bitwise Ideas Inc. Geolab User's Guide. New York: BitWise Ideas Inc., 1983.
122. Werner Gurtner. RINEX: The Receiver Independent Exchange Format Version 2. Berne, Germany: University of Berne, 1995.
123. Kang Jingfeng, "Establishing A-Order high accuracy reference network for GIS applications in Iowa", master thesis, Dept. of Civil Engineering, Iowa State University, 1997.

124. Digital and Kalman filtering: an introduction to discrete-time filtering and optimum linear estimation, Bozic, S. M., New York, Halsted Press, 1994.
125. Kalman filtering: theory and practice, Grewal, Mohinder S., Englewood Cliffs, N.J: Prentice-Hall, 1993.
126. Introduction to random signals and applied Kalman filtering, Brown, Robert Grover. New York: J. Wiley, 1992.
127. Estimation, control, and the discrete Kalman filter, Catlin, Donald E., New York, Springer-Verlag, 1989.
128. Forecasting, structural time series models and the Kalman filter, Harvey, A. C. Cambridge; New York: Cambridge University Press, 1989.
129. Mathematics of Kalman-Bucy filtering, Ruymgaart, P. A., Berlin; New York, Springer-Verlag, 1988.
130. White noise theory of prediction, filtering, and smoothing, Kallianpur, G., New York, Gordon and Breach Science Publishers, 1988.
131. An introduction to Kalman filtering with applications, Miller, Kenneth S., Malabar, Fla.: Krieger Pub. Co., 1987.
132. Lectures on Wiener and Kalman filtering, Kailath, Thomas., Wien; New York, Springer-Verlag, 1981.

133. Digital and Kalman filtering: an introduction to discrete-time filtering and optimum linear estimation, Bozic, S. M., London: E. Arnold, 1979.
134. Adaptive estimation algorithms, Levy, Larry James, 1971.
135. Variance of the state estimation error of an adaptive Kalman filter, Baker, Allan Eugene, 1967.
136. Estimation of covariance parameters for an adaptive Kalman filter, hellenbarger, Jerome Clair. 1966.
137. Kalman filtering and statistical process control, Crowder, Stephen V. 1986.
138. Kalman filtering theory, Balakrishnan, A. V., New York: Optimization Software, Inc., Publications Division, 1984.
139. Kalman filtering: theory and application, New York: IEEE Press, 1985.
140. Kalman filtering: theory and practice, Grewal, Mohinder S., Englewood Cliffs, N.J.: Prentice-Hall, 1993.
141. Kalman filtering with real-time applications, Chui, C. K., Berlin; New York, Springer-Verlag, 1987.
142. Evaluation of Preliminary Models of the Geopotential in the United States, D. A. Smith and D. G. Milbert, National Geodetic Survey, NOAA, SSMC3, 1315 East-West Hwy., Silver Spring, MD 20910

143. Bible, J.L. (1962). Terrain Correction Table for Gravity, Geophysics, Vol. 27, pp.715-718.
144. Pagiatakis, S.D. (1996). Towards a Refined Gravimetric Geoid Model for Canada 1996-1999, International Geoid Service Bulletin N. 5, pp. 31-39.
145. Sideris, M. (1993). Tests of a Gravimetric Geoid in GPS Networks, Surveying and Land Information Systems, Vol. 53, No. 2, pp. 94-102.
146. Wichiencharoen, C. (1982). The Indirect Effect on the Computation of Geoid Undulations, Report No. 336, Department of Geodetic Science and Surveying, OSU, The Ohio State University, Columbus, Ohio.
147. Dru A. Smith, Gravity and the Geoid at NGS, National Geodetic Survey, Presented at the 2000, Geodetic Advisor Convocation, Silver Spring, MD, April 11, 2000
148. Dennis G. Milbert, GPS Accuracy Before and After SA Removal, <http://www.igeb.gov/sa/diagram.shtml>, 2001, National Geodetic Survey, NOAA.
149. National Geodetic Survey (NGS) –CORS, 2003, <http://www.ngs.noaa.gov/CORS>

150. Indorante, S.J., R.L. McLeese, R.D. Hammer, B.W. Thompson, D.L. Alexander. 1996. Positioning soil survey for the 21st century. *J. Soil and Water Conservation* 51:21-28.
151. Valentine, K.W.G., W.C. Naughton and M. Nava. 1981. A questionnaire to users of soil maps in British Columbia: Results and implications for design and content. *Can. J. Soil Sci.* 61: 123-135.
152. GIS by ESRI, Arc/Info version 7, AML Commands, Environmental Systems Research Institute, INC. 1994.
153. Donna J. Peuquet, Geographic Information Systems, London, New York, Philadelphia, 1990.
154. The NGS GEOID Page, <http://www.ngs.noaa.gov/GEOID>, 2003
155. GEOID99, <http://www.ngs.noaa.gov/GEOID/GEOID99/>, 2003
156. GEOID96, <http://www.ngs.noaa.gov/GEOID/GEOID96/>, 1998
157. Global Positioning System Overview, 2003,
http://www.colorado.edu/geography/gcraft/notes/gps/gps_f.html