Field Pilot Results
for the
State of Iowa
Department of Transportation's
Linear Referencing System

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Prepared by

GeoAnalytics™

1716 Fordem Avenue
Madison, WI 53704
Phone: 608-241-7100
Facsimile: 608-241-7116
E-mail: office@geoanalytics.com
URL: http://www.geoanalytics.com

Dr. Alan Vonderohe
W10751 Wildwood Way, Route 3
Poynette, WI 53955
E-mail: vonderoh@gdinet.com

Iowa Department of Transportation

800 Lincoln Way
Ames, IA 50010
Phone: 515-233-7770
Facsimile: (515) 239-1828
E-mail: William.schuman@dot.state.ia.us

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Field Pilot Results for the Iowa DOT Linear Referencing System

1. Executive Summary

1.1. Background

In April 1999, the Iowa Department of Transportation (DOT) began a project to develop a Linear Referencing System (LRS) for the Department. A linear reference system's primary purpose is to improve Iowa DOT business workflows and decision-making by improving the integration of disparate data using the data's linear locations as the common link. The data's linear location is described in terms of a linear reference method (LRM). LRMs are used to locate transportation objects (signs, pavement) and events (crashes, traffic collection sections) relative to a position along a transportation feature (e.g. a roadway). Referencing transportation objects by milepost is an example of a DOT LRM. The DOT has identified six key LRMs used in the Department.

The purpose of the Iowa DOT LRS Development Project is to improve how the DOT manages and applies its LRMs by developing a linear reference system to integrate these methods and their associated business data. Specifically, there are five project objectives:

1. The LRS will provide improved data integration and access.
2. The LRS will provide improved accuracy of the features referenced to the road network.
3. The LRS will provide a way to linearly locate roadway data along all public roads in the State.
4. The LRS will help minimize redundancy in DOT database systems.
5. The LRS will help minimize data maintenance that is needed due to changes in the transportation network.

Iowa DOT contracted with GeoAnalytics, Inc. to provide counsel and facilitate Department decisions related to improved linear reference management. In addition, GeoAnalytics will provide technical support services for the testing and validation of LRS design decisions. The Project Team assigned to this project is composed of both GeoAnalytics and Iowa DOT staff members. A Project Steering Committee, composed of representatives from DOT Divisions, guides the Project Team.

1.2. Approach

This project has been broken into several phases. The Project Team successfully completed the first several phases that validated the need for the LRS and produced a comprehensive design of the LRS. Results from a project task called the Field Pilot are documented in this report. This project task is part of the Pilot phase of the project and also includes the System Pilot task. The Field Pilot Team is composed of Iowa DOT staff, and consultants from GeoAnalytics and their partners. The Field Pilot results will be used to meet Project Objective 2: "The LRS will provide improved accuracy of the features referenced to the road network."

To accomplish this objective, Iowa DOT must first improve the accuracy of the road network to which the features are referenced. Based on the LRS design, the essence of the road network is the linear datum. The linear datum is a fundamental and stable representation of the road network composed of anchor sections (sections of the road network) terminated by anchor points. Anchor points can be intersections or distinct and physically identifiable features on the roadway (a bridge expansion joint). Because the road network is based on the linear datum, Iowa DOT must begin accuracy assessments with the linear datum.

Therefore, the primary purpose of the Field Pilot was to determine which of several data collection methods would produce the accuracy required of the linear datum. In addition to the linear datum, the Pilot Team was to determine which collection methods would produce the relative accuracy required of Reference Posts (mileposts) and Station Markers (station posts and station stamps). The major goals of the Field Pilot are as follows:

1. Datum Design: Develop and test how to assign a linear datum (anchor sections and anchor points) to the road network.
2. Datum Collection (Survey): Collect a sample of the linear datum using all the pre-determined collection methods, and evaluate the viability of these methods based on several criteria. The primary criteria in the pilot should be accuracy, but cost, safety, and the ability to work on all roadways should also be considered.
3. Datum Adjustment: Develop and test how to manage measurement error and quantitatively document the error.
4. Reference Feature Collection: Collect reference post and station marker locations using pre-determined methods, and evaluate the viability of these methods based on relative accuracy.

1.3.Field Pilot Recommendations

The following recommendations describe which of the several data collection methods tested in the pilot could be used to produce the linear datum, reference post locations, and station marker locations (posts and stamps). These recommendations also include how to assign the datum to the roadway network and include what DOT business areas will be impacted by implementing the recommendations.

The Project Team will use the recommendations to modify the LRS design and then develop a full-scale LRS implementation plan with related cost estimates. These tasks are part of the remaining phases of this project. Cost estimates will include staff time, equipment, and services required to collect and maintain the LRS. As part of developing the cost estimate, the Project Team will analyze alternatives to implementing the data collection methods recommended. The Project Team will also analyze alternatives that may relax accuracy requirements in order to reduce costs.

The Project Team will then deliver the implementation plan with related cost estimates to the LRS Steering Team for decision-making and approval. The approval will complete this project and the plan will then be used to begin subsequent projects that implement the LRS.

1.3.1. Datum Design Recommendations

1. Anchor sections should be no more than five miles in length. Six miles is acceptable with a spanning measurement (a measure across two anchor sections).

   The average length of a primary anchor section, excluding ramps, is estimated to be 4 miles in length. Approximately 3000 primary anchor sections will be created plus 2000 ramp anchor sections. Some primary anchor sections will also include portions of roads from other systems. This will minimize the number of datum objects Iowa DOT will have to maintain when full implementation is achieved.

2. Redundant measures are made for each anchor section. The statistical adjustment software developed for the pilot will be modified for use in prototyping.

3. Data visualization software using GIS will be developed to place datum objects and to populate the database.

4. Data on changes to the system will be tracked so that estimates on maintenance of the system can be made.

1.3.2. Datum Collection Recommendations

1. Orthophotos with existing roadway inventory data will be used for initial datum creation. The best orthophotos available should be used. In most rural counties this will mean USGS orthophotos with a pixel size of 1 meter. The pilot tested orthophotos with a pixel size of 2 feet or less which did meet accuracy requirements. Since USGS orthophotos were not tested this method was not tested in the pilot, accuracy data will need to be determined. It will probably be necessary to relax accuracy requirements to use USGS orthophotos.

2. Where data is not available (USGS orthophotos may be 10 years old), DMI / DGPS will be used to gather the data. This will not be done with the new video log van because of logistic problems. The vehicles equipped with this technology will not include Inertial Navigation Systems (INS) because of the cost.

3. Ramps cannot be collected using the method identified in item 1. Two alternatives follow.
   a. Use DMI / DGPS to measure all ramps. If this method is used, anchor point accuracy will need to be relaxed.
   b. Use orthophotos to measure distances and kinematic GPS to locate anchor points. There will be some safety concerns with kinematic GPS and it might cost more than method a.

1.3.3. Datum Maintenance Recommendations

1. Roadway improvement design plans will be used for maintaining the Datum on the primary system.

2. DMI / DGPS measurements will be used as the redundant measure for the primary system.

3. Options for obtaining more current information from local governments will be explored during the Prototyping project.

1.3.4. Reference Feature Collection Recommendations

1. DMI / DGPS will be used for reference post location.

2. DMI / DGPS will be tested for use in collecting literal description feature locations (bridges and railroad crossings).

1.3.5. Organizational Recommendations and Issues

1. A formal LRS management structure, such as a board of directors, should be put in place.

2. Some increase in staff or reallocation of staff will be required for creating and maintaining the LRS. Estimates on staff levels will be determined in the remaining phases of the project.

    a. A LRS Manager Position will be needed.

    b. The Office of Transportation Data will require additional positions. They are the Office tasked with most Datum creation and maintenance activities. The System Monitoring Section will be responsible for most field activities. The System Management Section will be responsible for office based data collection processes.

    c. District Offices will be involved in anchor point monumentation, reference post placement, as built design plan, and design plan maintenance activities.

    d. Highway Division Staff will be involved in design plan maintenance activities.

    e. Information Technology support staff in both Applications Technology and Division Support Teams will be impacted

2. Overview

The goal of the Iowa DOT Linear Referencing System (LRS) Field Pilot Task is to demonstrate the implementation of the proposed and accepted field component of the LRS design. The Pilot must satisfy the goals and objectives outlined for the field component of the LRS Design and test a set of business processes against a set of methodologies for field data collection of the LRS. Ultimately, the Field Pilot Task provides feedback for refinement of the LRS design, selection of optimum methods for field data collection that meet the accuracy requirements of users. It also provides the basis for development and refinement of detailed procedures for LRS datum design, field data collection, and adjustment field measurements.

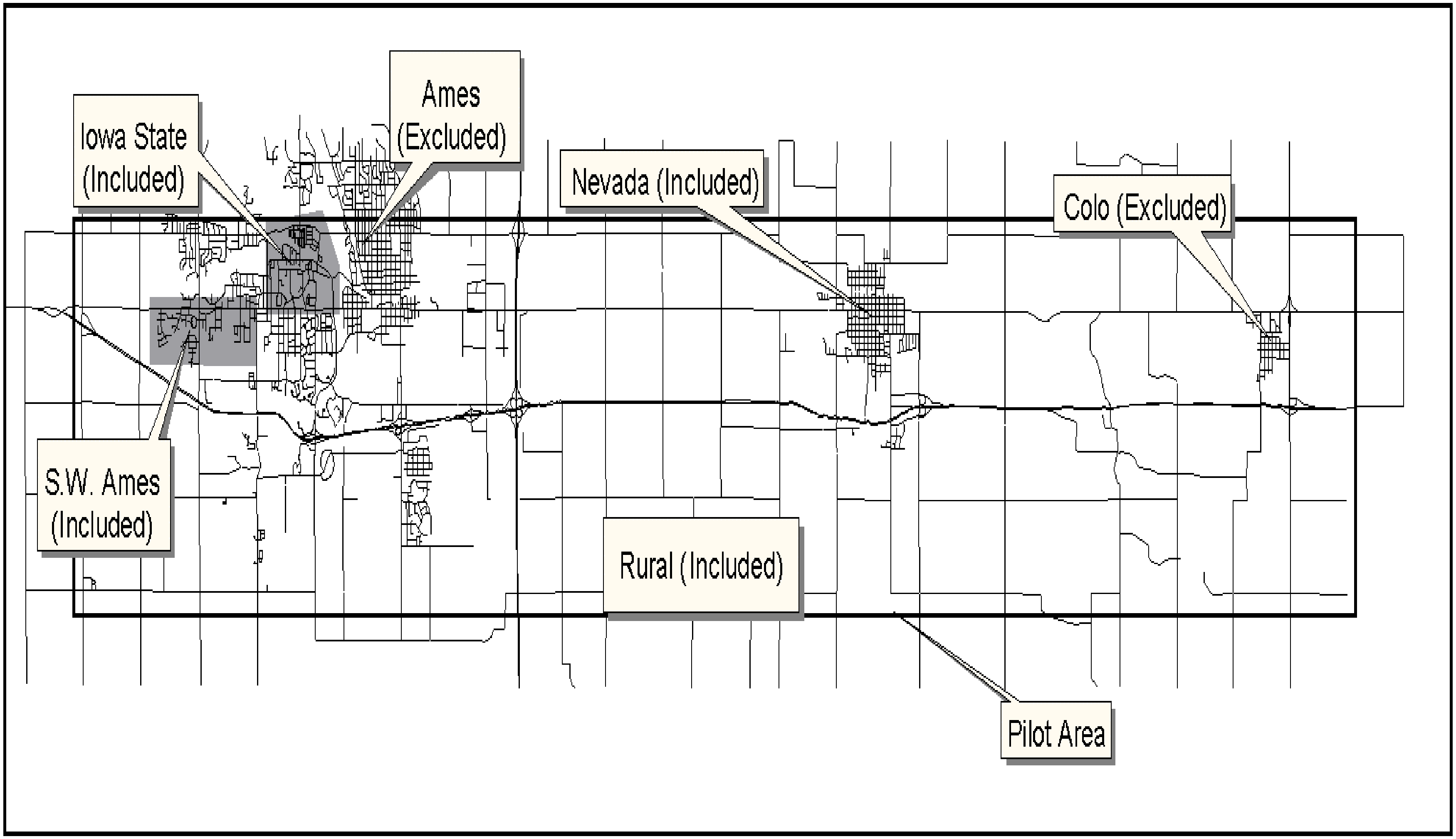
The LRS Field Pilot Project covers a 4X22 mile corridor in Story and Boone Counties in Central Iowa (see Figure 2-1). The Pilot includes all public roadways outside of municipal boundaries, all public roadways within the Town of Nevada, and selected public roadways within two areas in the City of Ames. Roadways within the Town of Colo are excluded from the Pilot, as are roadways within the City of Ames that are not within the two selected areas.

The Pilot includes 252 anchor sections and 462 anchor points, constituting the linear datum. These numbers include 10 anchor sections retired from the original design, 29 anchor sections added to the original design, and 41 anchor points added to the original design. Changes to the original design were required because the design was based upon a cartographic representation that included some errors of omission, some errors of commission, and some lack of currency. In addition, some errors in interpretation of the cartographic representation were made during the datum design process.

A number of methodologies were used to measure anchor section distances, anchor point span distances, and anchor point coordinates. In addition, some of these methodologies were used to measure anchor section offsets and coordinates of reference posts, station stamps and station posts. Reference data sets were selected, based upon their expected levels of relative accuracy, and comparisons were made among the various methodologies. Ultimately, the goal was to identify the optimum methodology, or mix of methodologies, which meet the accuracy requirements of the users of business data (i.e., ±10m at 90% confidence).

This document is structured according to five business processes appearing in the business process hierarchy in the document “A Logical Model for the State of Iowa Department of Transportation’s Linear Referencing System”: 1) design linear datum, 2) survey linear datum, 3) adjust linear datum, 4) place reference post, and 5) place station post. Each of these processes is described in terms of its purpose, the procedure used, and, where applicable, an evaluation of the measurement methodologies that were tested. These processes are evaluated by applying a set of criteria and then testing hypotheses, which state expected outcomes.

Figure 2-1 LRS Field Pilot Project Corridor



3. PILOT DEFINITION

3.1. Data and Process Scope

The scope of data and processes addressed during the Field Pilot task is best described by reference to components of the LRS Logical Design as referenced below.

3.1.1. Subsystems and Data Entities

The Pilot will create data for the following linear reference system subsystems and data entities:

- Linear Datum Management Subsystem: anchor point, anchor section, measurement methods, anchor point monument entities
- Reference Post Management Subsystem: reference post entity
- Stationing Management Subsystem: station marker entity

3.1.2. Linear Reference System Processes

Bold italic processes below are within scope of the Field Pilot. Position Linear Datum was targeted for the Field Pilot but is covered in the System Pilot.

1.2.2. Establish Linear Datum
- ***1.2.2.1. Design Linear Datum (Field)***
  - ***1.2.2.1.1. Determine First Order Datum Design***
  - ***1.2.2.1.2. Determine Second Order Datum Design***
  - ***1.2.2.1.3. Determine Third Order Datum Design***
- ***1.2.2.2. Conduct Linear Datum Survey (Field)***
  - ***1.2.2.2.1. Plan Linear Datum Survey***
  - ***1.2.2.2.2. Prepare Measurement Device***
  - ***1.2.2.2.3. Survey Linear Datum***
    - ***1.2.2.2.3.1. Place Anchor Point***
    - ***1.2.2.2.3.2. Monument Anchor Point***
    - ***1.2.2.2.3.3. Measure Anchor Point Span***
    - ***1.2.2.2.3.4. Determine Anchor Section Distance***
  - ***1.2.2.2.4. Adjust Linear Datum***
  - 1.2.2.2.5. Position Linear Datum (System)
- 1.2.2.3. Publish Linear Datum (System)

1.2.4. Establish Linear Reference Method
- ***1.2.4.1. Establish Reference Marker***
  - ***1.2.4.1.1. Place Reference Marker***
  - 1.2.4.1.2. Position Reference Marker
  - 1.2.4.1.3. Publish Reference Marker
- ***1.2.4.2. Establish Station Post***
  - ***1.2.4.2.1. Place Station Post***
  - 1.2.4.2.2. Position Station Post
  - 1.2.4.2.3. Publish Station Post

3.2. Measurement Methods Scope

The following measurement methodologies were tested:

1. Kinematic GPS for coordinates.
2. Videolog van DMI for distances.
3. Heads-up digitizing of low-resolution (2ft pixels) digital orthophotos for coordinates and distances. Two operators were used to provide independent redundant data sets.
4. Videolog van GPS/INS for coordinates and distances.
5. Heads-up digitizing of high-resolution (6 inch pixels) digital orthophotos.
6. Field inventory for distances.
7. GIMS cartography for coordinates and distances.
8. Clean cartography for coordinates and distances.
9. COGO of project plans for coordinates and distances.
10. Roadware van DMI for distances.
11. Roadware van GPS for coordinates and distances.

3.3. Method Selection Criteria Scope

The following criteria were used to evaluate these methodologies.

1. *Ability to register Linear Reference Method (LRM) traversal reference points along the datum.* Does the method support locating reference objects (e.g., reference posts, station posts) relative to anchor points by offsets along anchor sections? Criterion values are 'Yes' or 'No'.
2. *Cost.* What are both the one-time costs and the operating costs to Iowa DOT to apply the method? Costs are incremental; that is, costs are above and beyond current costs (e.g., current staff time, current equipment maintenance). Costs include investments in new data and upgrades to existing data, equipment, and processes. For one-time costs, labor is built into the data preparation and datum collection. Cost values are gross ranges: High (More than $1-2 million), Median ($100,000 to $1-2 million), or Low (under $100,000). Quantitative costs will be determined for the selected methods and provided in the Cost Estimate phase of the project.
3. *Accuracy (including repetition and redundancy).* The original datum design assumes that one-half the error in location of business data should be attributable to the measurement made to the reference object to locate the data of interest. The remaining error should be attributable to the LRS. Therefore, one of the original design criteria was that a reference post should be locatable along an anchor section to within ±5m at 90% confidence. This criterion reduces by calculation to ± 2.1m as a maximum allowable standard deviation in an anchor section distance (see Appendix 11.1). Accuracy estimates arose from two sources: 1) least squares adjustment of systems of redundant measurements, yielding the standard deviation of each anchor section distance as a by product and 2) comparison with reference data sets expected to be of higher accuracy. The following outlines how the accuracy criteria will be measured:
    - There will be three kinds of datum accuracy:
        - Results of least squares adjustments of redundant measurements.

    - Anchor section distance differences by comparison of a method to a reference data set.
    - Anchor point coordinate differences by comparison of a method to a reference data set.
  - For each kind of datum accuracy, the following metrics are compiled:
    - Mean (in meters, e.g., 5.0m).
    - RMS standard deviation (in meters; e.g., 2.5m).
    - Qualifier – number of rejected measurements (outliers). The rejections will be provided as a total count of rejections and as a percentage of total observations (e.g., 47 (19%)).
    - Qualifier – the reasons for the rejections. The reasons will be provided in the Process sections of this report.
  - For reference marker accuracy (input data is a difference in distance calculated by comparing data gathered by a measurement method with a reference data set; distance is measured along an anchor section from an anchor point):
    - Mean – same as the datum.
    - RMS standard deviation – same as the datum.
    - Qualifier – number of rejected measurements (outliers). Same as the datum.
    - Qualifier – the reasons for the rejections. Same as the datum.

4. *Safety*. The degree to which the method permits avoidance of damage or harm to people and property. The criterion values are:
   - High: fundamentally no risk.
   - Medium: moderate risk (persons are within vehicles).
   - Low: high risk; persons are exposed directly to traffic.
5. *Ability to apply to all road surfaces.* The confidence associated with using the method to interpret the location of anchor points and collect anchor section / anchor point span distances based on the road surface (gravel, dirt, paved). Criterion values are ‘Higher’, ‘Lower’.
6. *Ability to represent all road configurations.* The confidence associated with using the method to interpret the location of anchor points and collect anchor section and anchor point span distances based on roadway configurations (e.g., ramps and connectors, diverging and converging roadways, bi-directional roadways, divided roadways, etc.)? Criterion values are ‘Higher’, ‘Lower’.
7. *Ability to capture the curvilinear nature of the roadway*. The confidence associated with using the method to collect anchor section and anchor point span distances based upon the planimetric and vertical curvilinear shape of the roadway? Criterion values are ‘Higher’, ‘Lower’.
8. *Produces a cartographic by-product.* Whether the method generates cartography that can be used to support the position datum process (link the datum to DOT cartography). This cartography must meet the resolution and accuracy requirements of the DOT (multi-lane divided representation, ramp representation, topologically structured). Criterion values are ‘Yes’ or ‘No’.

9. *Produces timely datum objects.* Whether the method allows the creation of the actual datum (as opposed to proposed datums) before the roadway opens to traffic. Criterion values are:
   - No – the method does not, and could not even with changes in DOT processes.
   - Low Potential – the method does not, but could; however it would require significant changes in DOT processes.
   - High Potential – the method does not, but could; it would require some changes in DOT processes.
   - Yes – the method currently does.

3.4. Primary Pilot Benchmarks

The key hypotheses tested in this Pilot task are provided below.

Hypothesis 1: The accuracy requirements for development of the linear datum are high enough that changes in environmental and / or instrumental conditions, during a day's run with a DMI / GPS van, will require calibration of the instruments at both the beginning and end of each day.

Hypothesis 2: There is no significant difference between distances measured along centerlines of bi-directional roadways and centerlines of driven lanes on those roadways. That is, the business rules for anchor section representation and anchor section measurement do not conflict. "No significant difference" means a value less than or equal to a tolerance based upon the datum's design criteria for accuracy.

Hypothesis 3: GPS coordinates, captured at rates within the capacity of the instrumentation and within feasibility for data management, represent the curvilinear nature of the roadway sufficiently to meet the accuracy requirements for measurement of anchor section distances.

Hypothesis 4: GPS coordinates, captured on-the-fly with a moving vehicle, meet the accuracy requirements for anchor point coordinates.

Hypothesis 5: In flat areas, such as Story County, anchor section distances, measured by heads-up digitizing of high-resolution orthophotos, meet the accuracy requirements of the linear datum.

Hypothesis 6: Historical versions of the datum can be developed from plans and then related to the current roadway system.

Hypothesis 7: Existing Iowa DOT data collection processes (i.e., Roadware Van GPS and DMI, field inventory) can be leveraged to include data collection for the LRS (datum and reference objects).

Hypothesis 8: The business rules, described in Appendix 11.2, for anchor point selection and identification can be implemented in the field.

3.5. Methodology for Accuracy Assessment

Whenever possible, measurement data sets containing redundancy were checked initially for internal consistency and best estimates were generated for unknown parameters (i.e., anchor section distances and anchor point coordinates). Software was developed that implements the least squares method (Vonderohe, 1998) for generation of anchor section distances from data sets containing redundant distances. This software was used to assist with analysis of the videolog van DMI and GPS/INS (inertial system) distance data sets as well as the low-resolution orthophoto distance data set. Output includes not only best estimates for anchor section distances but also estimates for the standard deviations in those distances. The videolog van GPS/INS coordinate data set also contained redundant data. In this case, software was developed and applied which computed the mean and standard deviation of the coordinates at individual anchor points.

All datasets were compared to a set of reference data sets and their differences were analyzed using a collection of developed software and existing tools such as spreadsheets. The process included rejection of differences larger than three times the standard deviation from the mean of the differences being tested. In many cases, significant percentages of the data sets were rejected. The general reasons for rejection of data are found in each process section.

The reference data sets used in the analysis were 1) the kinematic GPS survey coordinates (expected to be accurate to better than 1 decimeter), 2) the least squares adjusted videolog van DMI anchor section distances (expected to be generally accurate and specifically accurate for anchor sections involving taper and gore points), 3) the least squares adjusted low-resolution orthophoto anchor section distances, and 4) the low-resolution orthophoto coordinates. The low-resolution orthophotos provided a comprehensive data set except for that portion of the Field Pilot task area in Boone County.

4. PROCESS: DESIGN LINEAR DATUM

4.1. Process Requirements

The field pilot tests the approach chosen to perform the process requirement "Design Linear Datum." This section describes the elementary processes, as well as the function and process under which they are defined.

Location Reference Maintenance (function)
Description: Ongoing activities necessary to maintain the LRS databases and the interfaces to Iowa DOT operational systems.

Design Linear Datum (process)
Description: Determine the specifications for locating and describing anchor points and sections and measuring anchor section distances.

Determine First Order Datum Design (elementary process)
Description: Select the optimal configuration for the datum. That is, select the numbers and locations of datum and reference objects and the measurements (and measuring device) that will be made using these. Specify the linear accuracy and allowable degree of error propagation of the datum (Vonderohe and Hepworth, 1998).

Determine Second Order Datum Design (elementary process)
Description: Determine the optimum accuracy of the anchor section and anchor point span measurements (Vonderohe and Hepworth, 1998).

Determine Third Order Datum Design (elementary process)
Description: Determine the optimal measurements necessary to improve or expand an existing datum (Vonderohe and Hepworth, 1998).

4.2. Process Approach

First and second order linear datum designs were performed for the Field Pilot task using the method described by Vonderohe and Hepworth (1998). Inputs to the design process included 1) a statement of the required accuracy of the linear locations of business data (i.e., ± 10 meters at 90% confidence) derived from the user needs assessment, 2) an estimate of the accuracy of videolog DMI measurements (i.e., ± 1 meter (68% confidence) pointing error at each end of a measured line plus an additional ± 1 foot per mile (68% confidence) measurement error) and 3) hardcopy cartographic representations of the layout of the roadway network. The DMI pointing error was estimated from data collected by repeat measurements of the calibration baseline along U.S. 30. The additional ± 1 foot per mile in a DMI measurement was derived from the manufacturer's specification for the device.

The design objective was to determine a datum configuration that minimized the number of datum objects while meeting the specified accuracy requirement for the locations of business data. Thus, it was critical to determine the length of the longest possible anchor section. The maximum anchor section length was determined by testing various combinations of anchor section distance and anchor point span measurements (that is, the design process is an iterative one). Once this determination was made, the hardcopy cartographic representations served as worksheets for layout of the datum (i.e., selection of anchor point locations and anchor sections) and assignment of unique identifiers for the datum objects. The business rules for anchor point selection served as guidelines (see Appendix 11.2) during datum design.

The datum information was transferred to a database and linked to the GIMS cartography to resolve errors in datum design (mismatched anchor point and anchor section IDs, anchor section gaps or overlaps, incorrect IDs, etc) and to help plan the field survey. The datum information also was transferred to fresh hardcopy maps to be used in the field during the survey of the linear datum.

No third order design was performed during the Field Pilot task, as all datum objects were being created for the first time. However, a business decision was previously made not to "feather" or merge measurements for new datum objects with those for existing datum objects during updates. Therefore, the third order design process is actually identical to that of first and second order design (e.g., future changes in alignment will effectively "stand alone" in their designs) and it is possible to generalize findings from the first and second order design experience to third order design.

4.3. Analysis of Findings

Appendix 11.1 contains both a derivation of the maximum allowable standard deviation in an anchor section distance ($\pm$ 2.1 meters) and a determination of the maximum anchor section length (six miles). The maximum allowable standard deviation in an anchor section distance is based upon the assumptions that 1) business data will be located by measurements to reference posts 2) half the allowable error in the locations of business data will be attributed to the measurements to reference posts and 3) the remaining half of the allowable error will be in the locations of the reference posts along their respective anchor sections. The maximum anchor section distance of six miles is allowable only when adjacent anchor sections have a spanning measurement (e.g., of 12 miles). If no spanning measurement is included, the maximum anchor section distance is five miles.

These criteria were applied to the roadway network, resulting in a design that included 421 anchor points, 233 anchor sections (to be measured in both directions or, on divided roadways, to be measured twice in the same direction), and nine anchor point spans (each to be measured once). Due to the configuration of both the roadway network and Field Pilot task corridor (elongated east-west), the density of datum objects varied by roadway system (see Appendix 11.5). The primary roadway system averaged one anchor section every four miles. The secondary roadway system averaged one anchor section every three miles. The municipal roadway system averaged one anchor section every 0.5 mile and the parks and institutional roadway system averaged one anchor section every 0.4 mile.

Given this analysis, the pilot team estimated the number of datum objects for the entire state (see Appendix 11.5). The team chose to create sub-totals for each roadway system (primary, secondary, municipal, and institutional) because of the anchor section density differences and to allow flexibility in LRS implementation. Ramp roadway configurations were estimated separately because of the distinct difference in business rules between converging and diverging roadway intersections and traditional cross street intersections (see Appendix 11.2). A datum for the current roadway system in Iowa would contain approximately 75,000 anchor sections, 129,000 anchor points, and 1800 anchor point spans. If roadway system alignment changes are approximately 1%-2% each year, annual maintenance to the datum could impact approximately 750-1,500 anchor sections, 1,300-2,600 anchor points, and 18-36 anchor point spans.

4.4. Findings

The maximum anchor section length of six miles, with a spanning measurement of 12 miles for adjacent anchor sections, is generally applicable and should be used as guideline for both full-scale implementation and updates to the datum. Without a spanning measurement, anchor section distances cannot exceed 5 miles.

A digital environment would be effective and efficient for creating datum objects and tagging them with identifiers. This creation process could place datum objects directly in the database.

Some GIS-based method for visualizing the configuration of the roadway network while designing the datum is essential. This visualization could range from aerial or satellite imagery, to a cartographic representation, to a sketch compiled from a literal description (such as might be available for an update (third order) design of local roads).

The software developed for adjustment of the linear datum (see Section 6) can be adapted to support datum design as well. This will require a few minor modifications.

The Office of Transportation Data, System Management should be responsible for performing these processes. This has not changed from the Physical Design Summary Document findings. The LRS project manager should contract initial datum survey. However, some System Management Office staff should participate to gain experience, validate data, and to make improvements in the processes and procedures for datum maintenance.

4.5. Practical Recommendations

Anchor sections will be no more than five miles in length. Six miles is acceptable with a spanning measurement.

A GIS-based visual environment will be developed for creation of datum objects using cartography and aerial photos. This environment will automate the placement of data in the Datum database.

The linear datum adjustment software will be modified for use in datum development and maintenance.

The LRS project manager will contract for the initial datum creation. The Office of Transportation Data, Systems Management will be closely involved with the process in order to facilitate their assumption of the maintenance activities.

5. PROCESS: SURVEY LINEAR DATUM

5.1. Process Requirements

The field pilot tests the methods chosen on how to perform the process requirement "Conduct Linear Datum Survey." This section provides a description of the elementary processes, as well as the function and process under which they are defined.

Location Reference Maintenance (function)
Description: Ongoing activities necessary to maintain the LRS databases and the interfaces to Iowa DOT operational systems.

Conduct Linear Datum Survey (process)
Description: Carry out the fieldwork necessary to build the linear datum.

Plan Linear Datum Survey (process)
Description: Determine the extent and order of the linear datum components to be established.

Prepare Measurement Device (process)
Description: Adjust and calibrate distance measuring device.

Survey Linear Datum (process)
Description: Establish field locations and distances for datum objects.

Place Anchor Point (elementary process)
Description: Using the datum design, locate and document Anchor Point in the field.

Monument Anchor Point (elementary process)
Description: Establish a permanent, recoverable monument at an anchor point location.

Measure Anchor Point Span (elementary process)
Description: Ascertain the linear, surface distance between two datum reference objects.

Determine Anchor Section Distance (elementary process)
Description: Allocate anchor span distances to anchor sections.

5.2. Process Approach and Analysis of Findings

This section describes the results of testing various data collection methods that could be used to survey, or collect, anchor point locations and anchor section distances. The Pilot staff used several methods to collect data. These methods were then analyzed; using the methods stated in section 3.3; to determine whether a method was appropriate for datum collection. A summary of the data results for each criterion is located in Table 5-1.

While all the criteria influenced the viability of each method, the Pilot primarily focused on gathering and analyzing the viability of each method's ability to meet the positional accuracy requirements of the LRS (Criteria 3). Locations for anchor points and / or distances for anchor sections where collected for each of the measurement methods. The collection processes varied with each of the methods and are, therefore, discussed with the findings for each method.

For each method, an overview is provided of the approach used to collect the datum data. The general findings from the accuracy analysis come next followed by the general findings from analyzing the remaining criteria. Conclusions reached from analyses are then summarized. In most cases, a detailed description of the accuracy findings follows these conclusions.

Accuracy is the most important criterion for method analysis in the Pilot. A brief overview of the accuracy statistics is described in the following paragraphs, which precedes the actual analysis of the pilot results. Accuracy data is created by comparing the anchor section and anchor point data set results collected for each of the methods to one or more reference data sets. Reference data sets provide a "ground truth" to which method results can be compared to determine if the method will produce accuracy results within Iowa DOT requirements. The reference data sets for anchor section distances included adjusted videolog van DMI anchor section distances and adjusted low-resolution orthophoto anchor section distances. The reference data sets for anchor point coordinates included kinematic GPS coordinates and low-resolution orthophoto coordinates.

For anchor section distances, three statistics were produced for each data set comparison: 1) the average discrepancy, 2) the standard deviation in the discrepancies, and 3) the root-mean-square (rms) discrepancy.

The average discrepancy is an indicator of bias between the reference data set and the data set being tested. A near-zero average discrepancy is good and means that there is very little systematic shift or difference between the data sets. A large average discrepancy means that there is some underlying cause that forces one of the data sets to be shifted, either positively or negatively, with respect to the other.

The standard deviation is an indicator of dispersion between the two data sets. A large value for the standard deviation means that there is great variation among the data. A small value for the standard deviation means that there is little variation among the data. That is, the greater the variation the less confident Iowa DOT can be in the suitability of the method for anchor section distance collection.

The rms discrepancy serves as the key statistic. The rms discrepancy takes into account both bias and variation. A value of 2.1m or less for the rms discrepancy means that the data sets agree to within the tolerance required for accuracy in the anchor section distances.

For anchor point coordinates, three statistics were produced for each data set comparison: 1) the average positional difference, 2) the standard deviation in the positional differences, and 3) the rms positional difference. The positional difference is the distance between the method's (x,y) location and the reference data set's (x,y) location. These statistics are computed similarly to those for anchor section distances.

To facilitate comparison of anchor section distances, software was written that finds matches in the data sets, does the differencing for each pair, and computes the statistics. The software then checks the absolute value of each anchor section difference against a tolerance of three times the standard deviation. Anchor section differences larger than this tolerance are rejected and the cycle is repeated until no anchor section distances are rejected. Each cycle of the process is an "iteration". A similar computer program was written for comparison of anchor point coordinates, with the rejection criterion being three times the standard deviation in either the x coordinate direction or the y coordinate direction. For each data set comparison, the number of iterations and the total number of rejected pairs is reported.

Table 5-1 Survey Datum and Place Linear Reference Results, by Method#

Criteria	Kinematic GPS	Videolog DMI	Videolog DGPS/INS	Low-Res Orthos	High-Res Orthos	Field Inventory	GIMS Carto	Clean Carto	Project Plans
1. Register LRM to Datum									
	No	Yes	Yes	No	No	Yes (possible)	No	No	Yes (Stationing)
2a. Incremental One-Time Datum Costs									
Summary	Medium	Medium	Medium	High	High	Medium	- (data quality issues)	Medium	- (limited availability)
Data Preparation	*Low (Basic Logistics Costs)*	*Low (Basic Logistics Costs)*	*Low (Basic Logistics Costs)*	*High+ (Statewide, Over $5 Million)*	*High+ (Statewide, Over $8 Million)*	*Low (Basic Logistics Costs)*	-	*Medium (but needs source for topology, divideds, ramps)*	-
Create Initial LRS	*Medium (AP only, 2 field trips, in low-mid $100K's)*	*Medium (Most Costly; shared with Videolog GPS); high $100K's or millions)*	*Medium (Most Costly; (shared with Videolog DMI, high $100K's or millions*	*Medium (High Cost; digitizing; in $100K's)*	*Medium (High Cost; digitizing; in $100K's)*	*Medium (Low Costs; data exists), in $10K's, but needs carto, too)*	-	*Low (Least Costly after initial cleanup; data exists), in $10K's*	-
Equipment	*Low-Medium (In $10K's or $100K's)*	*Low (If use existing and change process)*	*Low (If use existing and change process; may improve due to SA off)*	*Low ($20K-$70K; work-stations)*	*Low ($20K-$70K: (work-stations)*	*Low ($10K-$50K; low end work-stations)*	-	*Low ($10K-$50K; low end work-stations)*	-
Application Development	*Low (in $1000's)*	***Low (in $1000's)***	*Low-Medium (In $10K's or $100K's; must improve GPS application)*	*Low (In $1000's*	*Low (In $1,000's)*	*Low (In $1000's)*	-	*Low (In $1000's)*	-

Criteria	Kinematic GPS	Videolog DMI	Videolog DGPS/INS	Low-Res Orthos	High-Res Orthos	Field Inventory	GIMS Carto	Clean Carto	Project Plans
2b. Incremental Operating Datum Costs									
Summary	AP's Only	Most Costly	Most Costly	High Cost (limited availability)	High Cost (limited availability)	Moderate Cost	- (data quality issues)	Least Costly	Low Costs (limited availability)
Labor	*AP's only*	*Most Costly*	*Most Costly*	*High Cost*	*High Cost*	*Moderate Cost*	-	*Least Costly*	*Low Costs Requires geo-reference (limited availability)*
Equipment	*Low-Medium (See One-Time Costs above)*	*Low (See One-Time Costs above)*	*Low (See One-Time Costs above)*	*High (specialized, perhaps contract)*	*High (specialized, perhaps contract)*	*Low (See One-Time Costs above)*	-	*Low (See One-Time Costs above)*	*Low (In $10K's)*
Equipment Maintenance	*Low (In $1000's)*	*Low (Current costs)*	*Low (Current costs plus $1000's due to GPS capability)*	-	-	*Low (In $1000's)*	-	*Low (In $1000's)*	*Low (In $1000's)*
3a. Accuracy – Least Squares Adjustment of Anchor Section Distances and Anchor Point Spans									
Std. Dev. Of Unit Weight	-	1.064 1.043*	1.088	1.066	-	-	-	-	-
# Degrees of Freedom	-	239 201*	244	205	-	-	-	-	-
RMS Std. Dev. in AS	-	2.5m 2.3m*	3.9m	2.3m	-	-	-	-	-
# Rejects and % of Total	-	24 (5%) 79(16%)*	59 (11%)	69 (14%)	-	-	-	-	-

Criteria	Kinematic GPS	Videolog DMI	Videolog DGPS/INS	Low-Res Orthos	High-Res Orthos	Field Inventory	GIMS Carto	Clean Carto	Project Plans
3b. Accuracy – Anchor Section Distance Differences									
Mean	-	-	-0.9m	-1.5m	-1.3m	-0.8m	1.6m	0.7m	-33.7m (Small Sample)
Std. Dev.	-	-	5.6m	3.5m	1.8m	36.8m	18.6m	17.1m	20.9m (Small Sample)
RMS	-	-	5.7m	3.8m	2.2m	36.8m	18.6m	17.1m	39.7m (Small Sample)
# Rejects and % of Total	-	-	35 (19%)	47 (22%)	22 (26%)	30 (14%)	36 (15%)	39 (16%)	2 (25%) (Small Sample)
3c. Accuracy – Anchor Point Position Differences									
Mean	0.0m (expected)	-	3.3m***	0.9m**	0.8m***	-	12.0m***	11.9m***	18.8m*** (Small Sample)
Std. Dev.	<0.1m (expected)	-	2.2m***	0.5m**	0.5m***	-	6.4m***	6.1m***	21.8m*** (Small Sample)
RMS	<0.1m (expected)	-	3.9m***	1.0m**	0.9m***	-	13.6m***	13.4m***	28.8m*** (Small Sample)
# Rejects and % of Total	-	-	333 (32%)***	11 (16%)**	33 (21%)***	-	73 (17%)***	77 (18%)***	3(20%)*** (Small Sample)

Criteria	Kinematic GPS	Videolog DMI	Videolog DGPS/INS	Low-Res Orthos	High-Res Orthos	Field Inventory	GIMS Carto	Clean Carto	Project Plans
3d. Accuracy – Station Post Offset Distance Differences									
Mean	0.3m	-	-	-	-	-	-	-	-
Std. Dev.	6.0m	-	-	-	-	-	-	-	-
RMS	6.0m	-	-	-	-	-	-	-	-
3e. Accuracy – Station Stamp Offset Distance Differences									
Mean	-1.9m	-2.5m	-15.2m	-	-	-	-	-	9.3m
Std. Dev.	6.8m	5.0m	12.5m	-	-	-	-	-	11.0m
RMS	7.1m	5.6m	19.7m	-	-	-	-	-	14.4m
3f. Accuracy – Reference Post Offset Distance Differences									
Mean	-	30.0m	37.1m	-	-	-	-	-	-
Std. Dev.	-	16.2m	33.0m	-	-	-	-	-	-
RMS	-	34.1m	49.6m	-	-	-	-	-	-
4. Safety									
	Low	Medium	Medium	High	High	Medium	High	High	High
5. Ability to Apply to All Road Surfaces									
	Lower	Lower	Lower	Higher	Higher	Lower	Higher	Higher	Higher (Limited Availability)
6. Ability to Represent All Road Configurations									
	-	Higher	Lower	Lower	Lower	Higher	Lower	Lower	Higher (Limited Availability)
7. Ability to Capture Curvilinear Nature of Roadway									
Planimetric	-	Higher	Higher	Higher	Higher	Higher	Lower	Lower	Higher
Vertical	-	Higher	Lower	Lower	Lower	Higher	Lower	Lower	Higher

Criteria	Kinematic GPS	Videolog DMI	Videolog DGPS/INS	Low-Res Orthos	High-Res Orthos	Field Inventory	GIMS Carto	Clean Carto	Project Plans
8. Produces Cartographic by-product									
	Yes	No	Yes	Yes	Yes	Yes (if GPS in vehicles)	No	Yes	Yes
9. Produces Timely Datum									
	High Potential (Anchor Points only)	High Potential (change trip time)	High Potential (change trip time)	Low Potential (recreate once road in place)	Low Potential (recreate once road in place)	High Potential (change trip time)	No (data not conducive to datum design)	Low Potential (requires change outside of custodian scope)	High Potential (if use design plans, not as-builts; need extraction tool developed)

 # When possible relative comparisons across a row are provided.
** DMI measurements made while van was moving were deleted and treated as rejections.*
*** Reference data set is kinematic GPS coordinates.*
**** Reference data set is low-resolution orthophotos.*
+ The costs are if no orthophotography existed for the state of Iowa. However, the state and county government have a cooperative orthophoto program which results in costs to the state of approximately $1000-$5000 per county, where orthophotos are available. In areas where these is no county coverage Iowa state government also has DEMs that can be used if new photography is flown. This also reduces the costs substantially.

5.2.1. Kinematic GPS

Approach

Iowa DOT planned and performed a reconnaissance trip to mark anchor points. Skilled survey crews then collected the anchor point coordinates in the field using kinematic GPS.

Accuracy Criteria

The kinematic GPS survey provided a highly accurate partial set of anchor point coordinates (118). The horizontal coordinates in this data set are expected to be accurate to better than 1 decimeter at the one standard deviation level. This accuracy is well within the Iowa DOT's LRS accuracy requirements for anchor point locations. The full kinematic GPS data set included 37 coordinate pairs for taper points, witness points, monuments, and gore points that are not actual anchor points. These points were deleted from the data set. This reduced data set serves as a reference for comparison with the other methods. The data set was reduced further for comparison with the videolog van DGPS/INS method. This further reduction involved deletion of seven additional points that were observed at islands in the centers of cul-de-sacs.

Other Criteria

This method includes technology that can only be applied for anchor point collection. Anchor sections cannot be collected using this method. The method produces a cartographic by-product, and the technology can be used to collect the datum in a timely fashion. However, the field trips are expensive. Additional personnel with specialized skills are required, the field set up time adds additional costs, and the specialized equipment can be very expensive per field crew ($15,000 or more). This method also puts field crews in direct interaction with traffic.

Findings

Kinematic GPS meets the LRS positional accuracy requirements for anchor points. The field costs and the ability to collect only anchor point locations make the method less desirable for initial datum collection. It is a very viable method for operational datum collection where LRS location accuracy requirements must be met.

5.2.2. Videolog Van DMI

Approach

This data set contained measurements of anchor sections, typically driven in both directions for bi-directional roadways and twice in the same direction for divided roadways. It also included measurements of nine anchor point spans for additional redundancy, based upon the requirements of the initial datum design outlined in 5.1 above. While the method tested is the Videolog Van DMI, it is assumed that vehicles equipped with this technology will be able to produce very similar results.

The raw data files were reduced by the Center for Transportation Research and Education (CTRE) at Iowa State University. Data reduction included application of corrections for DMI calibration. The DMI was calibrated at the beginning and end of each day and differences in the two calibrations were prorated throughout the day using the timestamps in the data stream.

Subsequently, a modification was made to the data set according to an error discovered in the DMI calibration baseline. The baseline is between reference posts 154 and 155 on U.S. 30 eastbound. The inverse between the kinematic GPS coordinates of these two posts yields a distance of 5284.6 ft on the map projection. Dividing by the map projection scale factor over the Field Pilot Project area (0.999599) yields 5286.7 ft on the ellipsoid. Multiplying by the elevation factor over the Field Pilot Project area (1.000043 (see Appendix 11.6)) yields an on-the-ground straight-line distance of 5286.9 ft for the calibration baseline. Thus, all videolog DMI distances were multiplied by (5286.9/5280.0=1.001307) to account for the long baseline.

Accuracy Criteria

The adjusted videolog van DMI anchor section distances (see Section 6.3.1) served as a reference data set for comparison with other methods. See Appendix 11.3 for a statistical analysis of the data.

Other Criteria

A DMI is best for collecting complex intersection roadways, and captures the curvilinear nature of roads very well. However, the videolog van DMI was a difficult method to apply on gravel roads and dead ends. Paved roads usually have a painted centerline and it was not always possible to position the van over the end of the roadway. More importantly, field trips are expensive given staffing and equipment needs. Access to the videolog van will be difficult and most likely initial datum collection cannot use this method unless contracted. There is no cartographic by-product and this is why this method is often combined with GPS/INS technology. As with all fieldwork, safety is a concern but there is limited direct interaction between the staff and traffic. Finally, the method could collect datum locations in a timely fashion but this would require changes in priorities and practices of the van’s current use.

Findings

The Videolog Van DMI meets the Iowa DOT’s LRS accuracy requirements for anchor section distances. However, the field costs, technology availability, and no resulting cartography make this method less desirable for initial datum collection. It is a very viable method for initial datum collection on important priority roadways where meeting accuracy requirements are necessary. It is also very viable for collecting the datum along ramp-related roadways. This holds true for both initial and operational datum collection. This method should be complemented by GPS/INS technology to collect cartography.

5.2.3. Low-Resolution Orthophotos

Approach

Anchor points and anchor section distances were heads-up digitized by two independent operators on digital orthophotos having an on-the-ground 2ft pixel resolution. In addition, four anchor point spans were digitized by one of the operators. Low-resolution orthophotos were available for all of the Field Pilot Project area except the western most mile (Boone County).

Accuracy Criteria

Low-resolution orthophotos do not produce anchor section distances within the required LRS accuracies, but will most likely in areas where anchor points are clearly photo-identifiable. Low-resolution orthophotos can be considered to meet the accuracy requirements for anchor point coordinates.

Other Criteria

Initial costs can be very expensive if an orthophoto program is not in place and orthophotos are not available. Fortunately, Iowa DOT has a complete set of USGS orthophotos and a commitment to purchase orthophotos from local governments, and this makes this method a very viable option to consider. This method is also attractive because field-related costs are substantially less than other field-based methods (e.g., Videolog DMI, Field Inventory, etc). The equipment requirements are basic GIS workstations and safety is not a concern.

This method produces a cartographic by-product and includes a photographic image of the landscape. Such an image is rich with information that cannot be easily obtained from any other source. This method can be used to represent all roadways except the converging or diverging ends of ramp-like roadway configurations. The curvilinear nature of the roadway is adequately represented except in areas of severe elevation change (e.g., the Mississippi River Valley).

The biggest disadvantage of this method is the temporal quality of the image. In areas with frequent roadway improvements or roadway expansions this method would fall short. For operational datum collection, this method will be inadequate for producing timely datum updates because it would require re-flying the areas of roadway improvement after some basic essence of the roadway is constructed.

Findings

Low-resolution orthophotos generally meet the desired LRS accuracy requirements. This is a desirable method for initial datum collection except for ramp-like roadway configurations, areas of significant elevation change, and areas where roadway construction is significant and frequent and where orthophotography is relatively old. This method may not be appropriate for datum collection maintenance unless Iowa DOT is able to acquire recent aerial images of changing areas.

Description of Accuracy Findings
The adjusted low-resolution orthophoto anchor section distances (see Section 6.3.3) were compared to the adjusted videolog van DMI anchor section distances. The adjustment of the anchor section distances combined the observations of the two operators to produce single distance values for each anchor section. Thus, the comparison of anchor section differences could not be separated by operator. The comparison statistics appear in the first row of Table 5-2. Rejected differences as large as 100m were common. The large majority of the rejected differences occurred at ramps where taper points were easily identifiable in the field but very ambiguous on the orthophoto. Other examples of rejected differences include temporal problems, where the 1995 orthophoto showed terminated roadways that had later been extended and then measured by the videolog van DMI.

The statistical results indicate that the low-resolution orthophotos do not meet the accuracy requirement of 2.1 meters in an anchor section distance (the rms difference of 3.8 meters). However, it should be considered that some of the distance differences contributing to the 3.8m rms might be along ramps that include ambiguous anchor points on the orthophotos. The orthophoto should be expected to produce better results on residential streets and in areas where anchor points are clearly photo-identifiable.

Table 5-2 Low-Resolution Orthophoto Statistics

Reference Method	Object	Initial Sample	Iterations	Number Rejected	Mean	Std. Dev.	RMS
Videolog Van DMI	Anchor Section Distances	208 pairs	11	47(23%)	-1.5m	3.5m	3.8m
Kinematic GPS Coords.	Anchor Point Coords. Operator 1	65 pairs	7	12 (18%)	1.0m	0.6m	1.1m
Kinematic GPS Coords.	Anchor Point Coords. Operator 2	68 pairs	5	11 (16%)	0.9m	0.5m	1.0m

The low-resolution orthophoto anchor point coordinates were compared to the kinematic GPS anchor point coordinates. Because two different operators independently collected the low-resolution orthophoto data, comparisons were done for each of the operators' data sets. The comparison statistics appear in the last two rows of Table 5-2. For rejected coordinates, differences of 100m were common for both operators. The large differences are often explained by temporal changes (given the date of photography for the orthophotos; see Figure 5-1) or by differences in interpretation of public / private roadways, for example, parking lots (see Figure 5-2 and Figure 5-3). The results indicate there is no significant difference in the data collected between the operators and, therefore, their data can be combined for anchor point accuracy statistical analysis.

Statistical analysis indicates that low-resolution orthophotos can be considered to meet the accuracy requirements for anchor point coordinates. The RMS positional difference for operator 1 is 1.1m and for operator 2 it is 1.0m. This allows anchor points to be located to within 1.0 meters on the ground, which is the pointing error used in design of the datum for the Field Pilot task.

See Appendix 11.4 for a deeper statistical analysis of the low-resolution orthophoto data.

5.2.4. Videolog Van DGPS/INS

Approach

This data set contained measurements of anchor sections, typically driven in both directions for bi-directional roadways and twice in the same direction for divided roadways. It also included measurements of nine anchor point spans for additional redundancy, based upon the requirements of the initial datum design. The data set also included horizontal coordinates for anchor points. All of these data were collected simultaneously with the videolog van DMI data. While the method tests the Videolog Van DGPS/INS technology, it is expected that any vehicle equipped with similar technology will produce similar results.

The raw data files were reduced by CTRE. The selected data reduction method included differentially corrected GPS coordinates using a base station and inertial system (INS) smoothing in coordinate gaps. The GPS base station at the Iowa State University Soil Tilth Laboratory was ultimately used due to suspected multipath errors in the Iowa DOT base station data. Preliminary processing of the raw data files produced ASCII files of latitude / longitude coordinates for anchor points and latitude/longitude coordinate strings for anchor section distance and anchor point span measurements. CTRE personnel imported these coordinate files into MGE. MGE functionality was used to convert these coordinates to the Iowa DOT system and to compute linear distances from the coordinate strings. An elevation factor was applied to the ellipsoid distances to obtain ground distances (see Appendix 11.6).

Accuracy Criteria

Videolog van DGPS/INS distance measurements do not produce anchor section distances that meet the accuracy requirement for surveys of the linear datum. Videolog van DGPS/INS coordinates are not adequate for establishment of anchor point coordinates. Typically, DGPS-based collection is attractive because the method has been proven elsewhere to produce acceptable cartographic locations. Unfortunately, the quality obtained on this Pilot is below expectations.

Other Criteria

The results of other method selection criteria were also compiled. As with all field trip methods, staffing and equipment make this method more expensive. The method is less desirable for gravel roads, and due to loss of the GPS signal, less desirable in hilly terrain or in areas with tall buildings. The method will adequately capture the curvilinear nature of roadways but will introduce error because of the quality of data obtained in the vertical dimension. For these reasons this method is often complemented with a gyro and or DMI technology (e.g., the videolog van).

Safety is a concern but there is limited direct interaction between the staff and traffic. Access to the videolog van will be difficult and therefore initial datum collection cannot use this method unless contracted. Finally, the method could be used to collect datum locations in a timely fashion but this would require changes in priorities and practices of the van's current use.

Findings

In conclusion, the Videolog Van GPS/INS does not meet the LRS accuracy requirements. This method is less desirable for initial datum collection because of field costs, and availability. It is a viable method for initial or operational datum collection as a complement to DMI methods.

Description of Accuracy Findings

Prior to comparison of the videolog van DGPS/INS distances to any reference data set, the reduced distances were adjusted by least squares (see Section 6.3.2). The adjusted videolog van DGPS/INS anchor section distances were compared to the adjusted videolog van DMI anchor section distances. The comparison statistics appear in the first row of Table 5-3. For rejected distances, differences as large as 25m were common. The sign of the mean discrepancy of –2.4 meters (DMI minus DGPS/INS) might seem contrary to intuition. That is, since the GPS/INS distances are computed along chords and the DMI distances are measured along arcs, common sense suggests that the DGPS/INS distances should be shorter than the DMI distances. However, the long DGPS/INS distances can be explained by the INS smoothing in coordinate gaps. This process often produced distances that were overly long (see Figure 5-4, Figure 5-5, and Figure 5-6). The rms of 5.1m in the distance differences indicates that videolog van DGPS/INS distances measurements do not meet the accuracy requirement for surveys of the linear datum.

Table 5-3 Videolog Van DGPS/INS Statistics

Reference Method	Object	Initial Sample	Iterations	Number Rejected	Mean	Std. Dev.	RMS
Videolog Van DMI	Anchor Section Distances	212 pairs	5	17(8%)	-2.4m	4.5m	5.1m
Low-Res Ortho Distances	Anchor Section Distances	188 pairs	8	35 (19%)	-0.9m	5.6m	5.7m
Kinematic GPS Coords.	Anchor Point Coords. (full set)	178 pairs	6	22 (12%)	2.2m	1.1m	2.5m
Kinematic GPS Coords.	Anchor Point Coords. (ave. set)	66 pairs	7	13 (20%)	1.3m	0.8m	1.5m
Low-Res Ortho Coords. Operator 2	Anchor Point Coords. (full set)	1031 pairs	12	333 (32%)	3.3m	2.2m	3.9m
Low-Res Ortho Coords. Operator 2	Anchor Point Coords. (ave. set)	433 pairs	11	156 (36%)	2.2m	1.4m	2.6m

Figure 5-1 Anchor Sections 1200, 1201, 1202

Figure 5-2 Anchor Section 1162

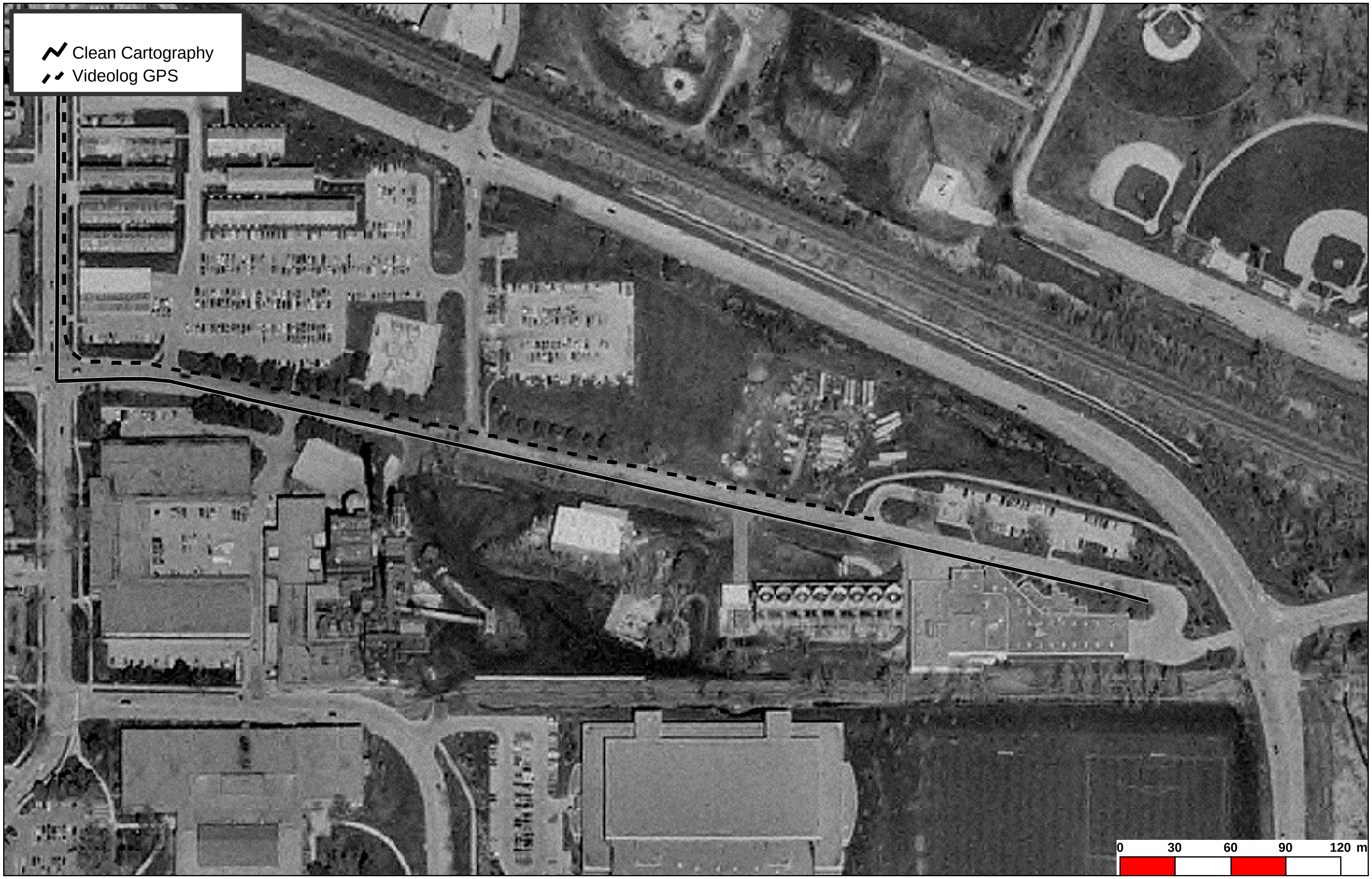


Figure 5-3 Anchor Section 1076

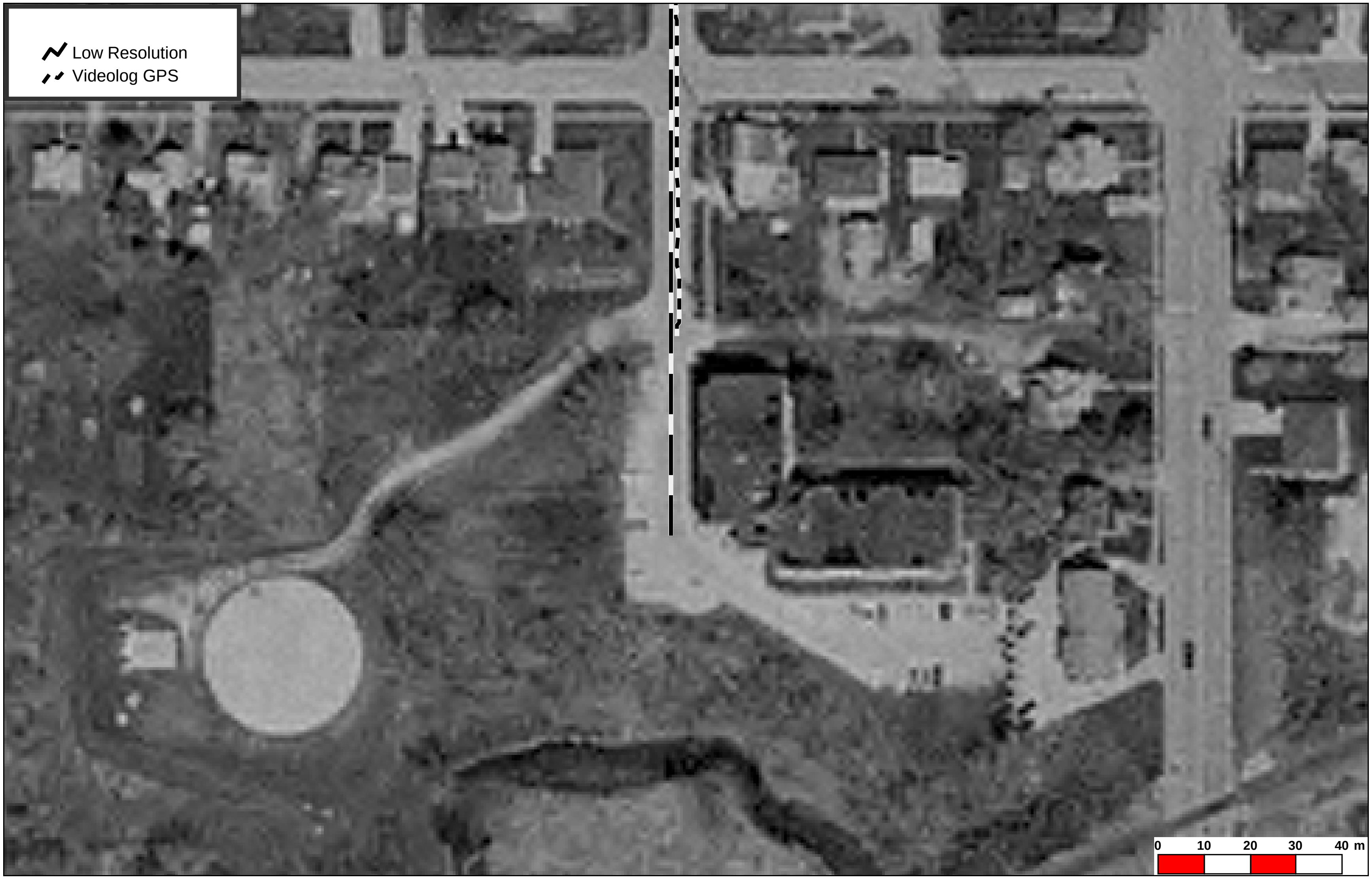


Figure 5-4 Two Redundant Measurements of Anchor Section 1167

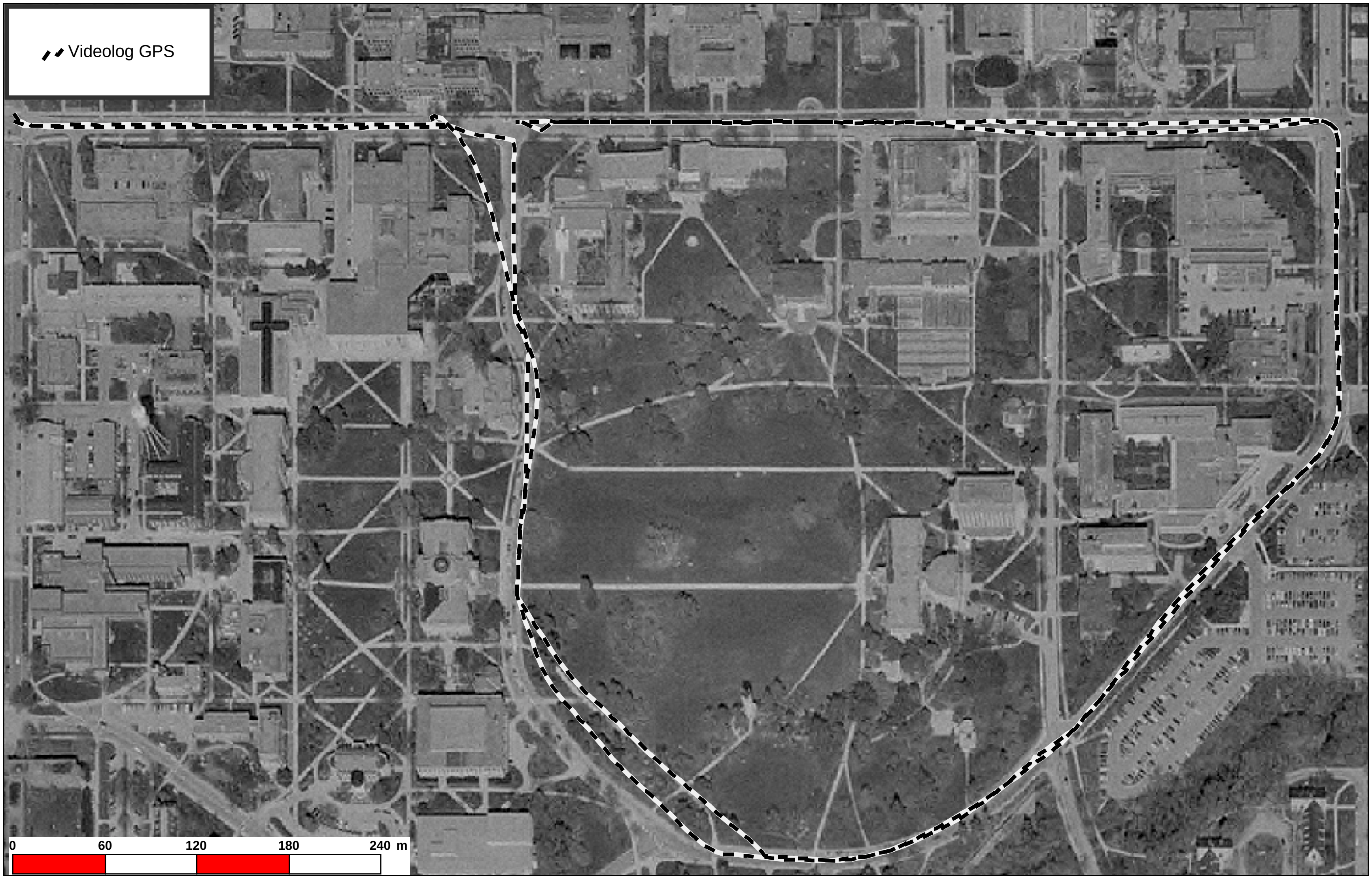


Figure 5-5 Videolog DGPS Measurements in Nevada

Figure 5-6 Videolog DGPS Measurements between Nevada and Colo along US 30

The adjusted videolog van DGPS/INS anchor section distances were also compared to the adjusted low-resolution orthophoto anchor section distances. The comparison statistics appear in the second row of Table 5-3. For rejected distances, differences as large as 200m were common. These large differences were invariably at ramps, at locations where changes in the roadway system had occurred since the orthophotos were made, or at locations where interpretation of the endpoints of roadways were different between the two methods (see Figure 5-2, Figure 5-3, and Figure 5-7). The rms discrepancy of 5.7m is larger than that for comparison with the videolog van DMI distances, indicating no improvement in the reliability of the videolog van DGPS/INS distance measurements.

The coordinates of anchor points measured by videolog van DGPS/INS were compared to those measured by kinematic GPS. Because a significant number of anchor points had more than one measurement of their coordinates by videolog van DGPS/INS, two forms of analysis were done. In the first, the full set of videolog van DGPS /INS measurements was used. That is, each coordinate pair measured by videolog van DGPS/INS was compared to the corresponding coordinate pair measured by kinematic GPS. The comparison statistics appear in the third row of Table 5-3. For rejected locations, positional differences as large as 50m were common. Some of the larger differences can be explained by anomalies in the videolog van DGPS/ INS data (see Figure 5-8). The average difference in the X direction for this data set was –0.3m and the average difference in the Y direction was 0.1m, indicating the presence of very little bias between the two data sets. However, the rms discrepancy of 2.5m between the two data sets indicates that videolog van GPS/INS coordinates are not adequate for establishment of anchor point locations.

In the second analysis, multiple videolog van DGPS/INS coordinate measurements at individual anchor points were averaged and the average values of the coordinates were assigned to the anchor points. The comparison statistics appear in the fourth row of Table 5-3. For rejected locations, positional differences as large as 40m were common. The rms positional discrepancy of 1.5m indicates that averaging the DGPS/INS coordinates of anchor points prior to analysis improves the overall accuracy.

The coordinates of anchor points measured by videolog van DGPS/INS were also compared to those measured by operator 2 on the low-resolution orthophotos. Operator 2 was known to have consulted field notes more often during the measurement process than operator 1. The low-resolution orthophoto coordinates of operator 2 were, therefore, thought to be more reliable than those of operator 1. Once again, two forms of analysis were performed due to the presence of multiple coordinate measurements for individual anchor points in the videolog van DGPS/INS data set. In the first analysis, each coordinate pair measured by videolog van DGPS/INS was compared to the corresponding coordinate pair measured on the low-resolution orthophotos. The comparison statistics appear in the fifth row of Table 5-3. For rejected locations, positional differences as high as 300m were common. The rms positional discrepancy is 5.3m.

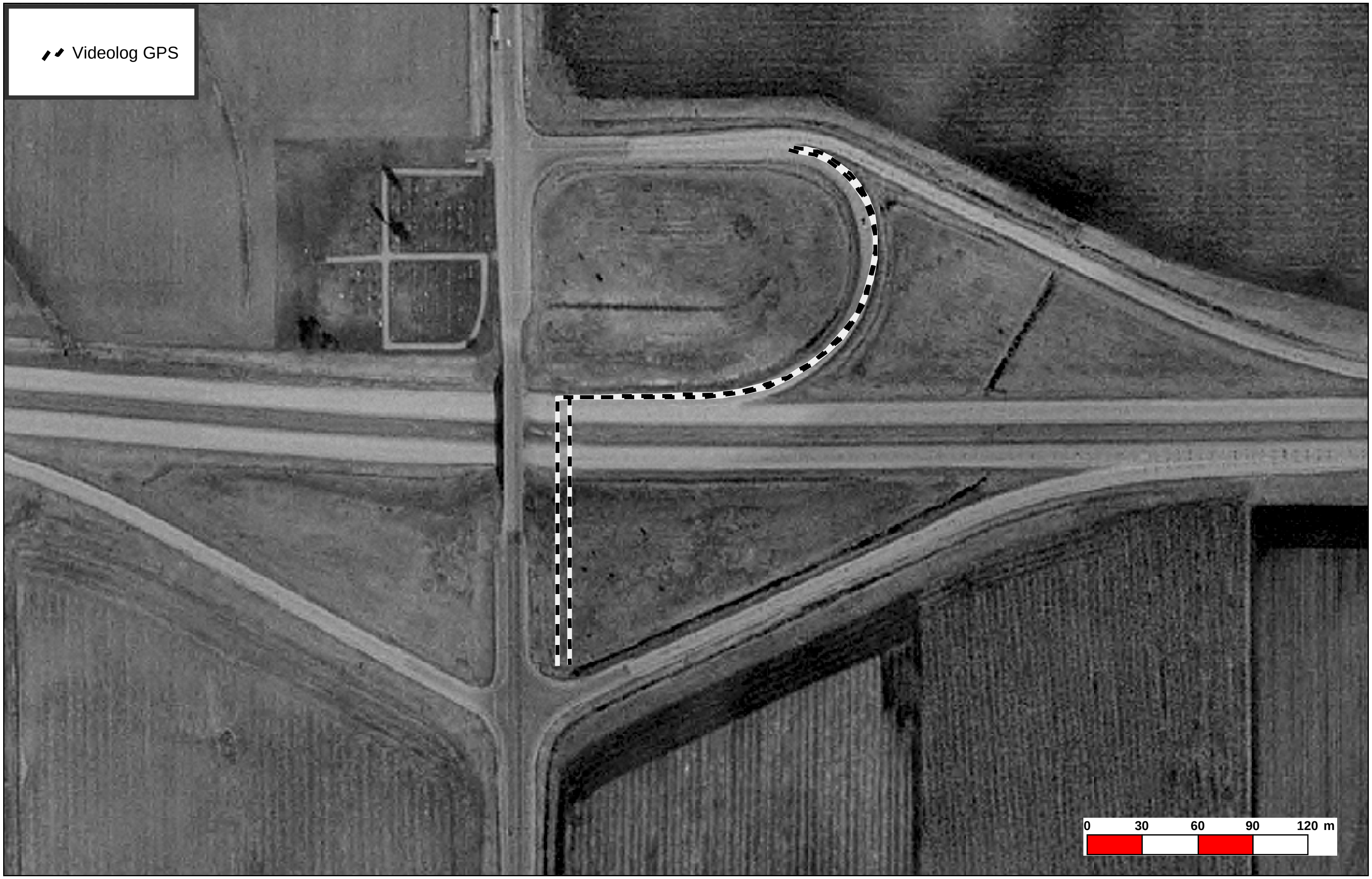
In the second analysis, multiple videolog van DGPS/INS coordinate measurements at individual anchor points were averaged and the average values of the coordinates were assigned to the anchor points. The comparison statistics appear in the sixth row of Table 5-3. For rejected locations, positional differences as large as 200m were common. The rms positional discrepancy is 2.6m.

The comparisons to the low-resolution orthophoto coordinates yielded slightly coarser results than the comparisons to the kinematic GPS survey. In any case, the videolog van DGPS/INS coordinates are not adequate for establishment of anchor point coordinates.

A significant number of anomalies were present in the videolog van DGPS/INS data set. Some of these were probably caused by multipath or weak satellite geometry due to obstructions in urban areas (Figure 5-5). Some might have been caused by the manner in which the data reduction software interpolates or "fills in" with inertial data when GPS data are missing in the data stream (see Figure 5-4). Other anomalies have no obvious explanation, but might arise from data reduction methods involving filling in GPS data using INS (see Figure 5-8).

Figure 5-7 Anchor Section 1081

Figure 5-8 Two Redundant Measurements of Anchor Section 1157



5.2.5. High-Resolution Orthophotos

Approach

Anchor points and anchor sections were heads-up digitized on digital orthophotos having an on-the-ground 6 inch pixel resolution. The geographic extent of the high-resolution orthophotos was much less than that of the low-resolution orthophotos, resulting in far less anchor points and anchor sections to compare. The high-resolution orthophotos were expected to yield better accuracies than the low-resolution orthophotos. Therefore, it would seem that the high-resolution orthophotos should serve as a better reference data set. However, their extent was so limited, that the resulting sample size would be too small to yield meaningful results.

Accuracy Criteria

The high-resolution orthophotos meet the accuracy requirements for survey of the linear datum for both anchor section distances and anchor point coordinates. This conclusion, however, is qualified by small sample size and by the fact that the data were, by necessity, compared to measurements that were expected to be less accurate.

Other Criteria

This method has the same results as the low-resolution orthophotos. An additional advantage of this method is the increased resolution found for locating more detailed roadway configurations and the indirect value in non-roadway information it provides. An additional disadvantage of this method is its limited availability.

Description of Accuracy Findings

The high-resolution orthophoto anchor section distances were compared to the adjusted videolog van DMI distances. The comparison statistics appear in the first row of Table 5-4. For rejected distances, differences as large as 100m were common. The rms difference of 2.2m is within 0.1m of the accuracy requirement for anchor section differences. The high-resolution orthophoto anchor section distances were also compared to the adjusted low-resolution orthophoto anchor section distances. The comparison statistics appear in the second row of Table 5-4. For rejected distances, differences as large as 30m were common. The rms difference of 1.8m is within the accuracy requirement of 2.1m. Therefore, the high-resolution orthophotos meet the accuracy requirement for anchor section distances.

Table 5-4 High-Resolution Orthophoto Statistics

Reference Method	Object	Initial Sample	Iterations	Number Rejected	Mean	Std. Dev.	RMS
Videolog Van DMI	Anchor Section Distances	84 pairs	11	22(26%)	-1.3m	1.8m	2.2m
Low-Resolution Orthos	Anchor Section Distances	80 pairs	10	15 (19%)	-0.7m	1.6m	1.8m
Kinematic GPS Coords.	Anchor Point Coords.	21 pairs	2	1 (5%) 4 (20%)*	13.3m 1.7m*	32.0m 1.5m*	34.0m 2.2m*
Low-Resolution Orthos	Anchor Point Coords. Operator 2	157 pairs	7	33 (21%)	0.8m	0.5m	0.9m

* After rejecting three more large differences

The high-resolution orthophoto anchor point coordinates were compared to the kinematic GPS anchor point coordinates. The comparison statistics appear in the third row of Table 5-4. The small sample of 20 contained three differences that were atypically large even though they fell within three standard deviations of the mean coordinate differences in both X and Y. These three atypical differences were deleted to produce the results marked by an asterisk in the table. If the standard deviation of 1.5m in the positional differences is taken as the indicator of accuracy, then the high-resolution orthophotos come within 0.5m of the accuracy requirement for coordinates of anchor points. This conclusion is supported by further comparison of the high-resolution orthophoto anchor point coordinates to the low-resolution orthophoto anchor point coordinates of operator 2. Statistics for this comparison appear in the fourth row of Table 5-4. The rms difference of 0.9m is within the accuracy requirement of 1.0m.

5.2.6. Field Inventory

Approach

Field inventory distances for anchor sections were extracted from base records. Where GIMS cartographic strings had been split to create cartographic representations of anchor sections, a procedure was developed for summing and proportioning the associated field inventory distances.

Accuracy Criteria

Comparison of field inventory anchor section distances to reference sets indicates that in some cases field inventory distances may be useful for population of anchor section distances, but that in other cases there are significant differences. Consequently, the overall statistical analysis rejects field inventory as a method that is comprehensively appropriate for survey of the linear datum.

Other Criteria

While the Field Inventory collection methods were not performed to collect data in the Pilot, it is assumed the method would be similar to the videolog van DMI. A DMI is best for collecting complex intersection roadways, and captures the curvilinear nature of roads very well. The DMI is difficult to apply on gravel roads and dead ends. There is no cartographic by-product and this is why this method is often combined with GPS/INS technology. Safety is a concern, but in most cases there is no or limited interaction directly between staff and other vehicles.

As with all field-based methods, the costs to staff and equip vehicles can be very high to initially collect the datum and update it over time. However, Iowa DOT should consider using the existing field inventory distance data to initially populate the datum. Given the accuracy findings described below, the data would need to be compared to an independent source to validate distance accuracies.

Operating costs would be much less if Iowa DOT could leverage existing field inventory processes. If this is possible, Iowa DOT will need to adjust the processes to improve some of the accuracy results. This method could also be applied to collect the datum to meet temporal requirements, but would require changes in current field collection practices and priorities at Iowa DOT. Field staff would need to collect data on all new roadways when the final alignment is defined in the field and the surface is passable (does not need to be a final surface). Therefore, these data would need to be collected during construction of these facilities.

Findings

The related field costs and the fact that the method does not produce cartography make this method less desirable for initial datum collection. Using the existing field inventory distances for initial datum collection is a viable method, but will require an independent validation source. This is because field inventory, while in most cases have distances that meet the LRS accuracy requirements for datum distances, include a significant number of distances that do not. Given Iowa DOT has an existing process in place that collects field inventory distances, this method is desirable for operational datum collection. However, existing processes will need to be evaluated to improve accuracy results. This method should be complemented with DGPS/INS technologies to collect the related cartography of new roadways.

Description of Accuracy Findings

The field inventory anchor section distances were compared to the adjusted videolog van anchor section distances. The comparison statistics appear in the first row of Table 5-5. For rejected distances, differences as large as 200m were common. The field inventory anchor section distances were also compared to the adjusted low-resolution orthophoto anchor section distances. The comparison statistics appear in the second row of Table 5-5. For rejected distances, differences as large as 200m were common.

Table 5-5 Field Inventory Statistics

Reference Method	Object	Initial Sample	Iterations	Number Rejected	Mean	Std. Dev.	RMS
Videolog Van DMI	Anchor Section Distances	225 pairs	10	42(19%)	-0.8m	36.8m	36.8m
Low-Resolution Orthos	Anchor Section Distances	210 pairs	6	22 (10%)	-3.7m	36.0m	36.1m

The rms differences of 36.8m and 36.1m indicate that field inventory distances do not meet the accuracy requirement for survey of the linear datum. However, further analysis indicates that many field inventory distances are more accurate than indicated by the full sample. Figure 5-9 and Figure 5-10 illustrate the distribution of distances differences across the two comparisons. These two figures also illustrate corresponding normal distributions. The large spikes within 0.5 standard deviations of the mean indicate that there are a significant number of field inventory distances that come close to agreement with the reference data sets. This suggests that with some appropriate (and perhaps minor) adjustment of the field inventory data collection process, this method might yield useful results in some cases for survey of the linear datum.

Figure 5-9 Distribution of Videolog Van and Field Inventory Distance Differences

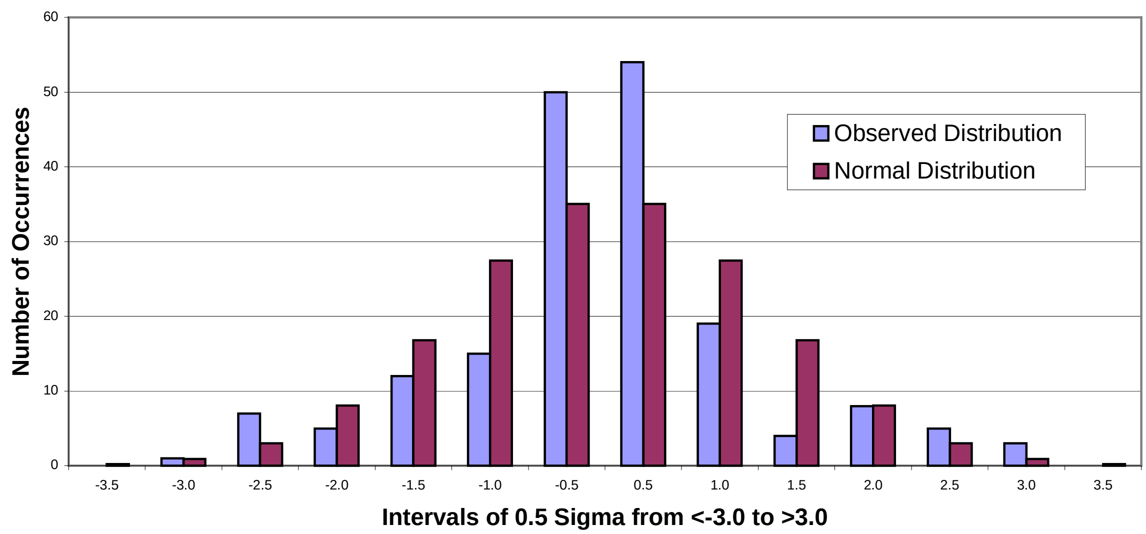
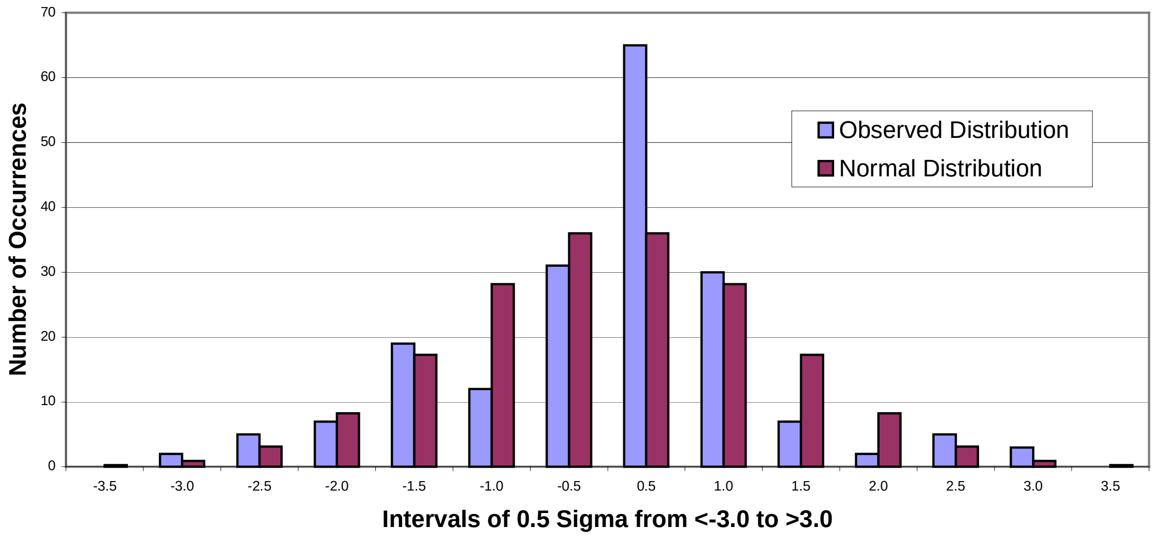


Figure 5-10 Distribution of Low-Resolution Orthophoto and Field Inventory Distance Differences



5.2.7. GIMS Cartography

Approach

After GIMS records were tagged with anchor section identifiers, anchor section distances were derived from GIMS cartography by first calculating string lengths and then summing them by anchor section. Anchor point coordinates were also extracted from GIMS cartography. The procedures were executed with Geomedia Pro and MS Access.

Accuracy Criteria

The GIMS cartography does not meet the accuracy requirements for survey of the linear datum, for either anchor section distances or anchor point coordinates.

Other Criteria

The cartography is not topologically structured and it does not contain divided roadways. These data were critical to the Pilot testing. Consequently, criteria other than accuracy were not analyzed for this method See the Clean Cartography Method (Section 5.2.8).

Conclusions

GIMS cartography in its current state is not a viable method.

Description of Accuracy Findings

The GIMS cartography anchor section distances were compared to the adjusted videolog van DMI distances. The comparison statistics appear in the first row of Table 5-6. For rejected distances, differences as large as 100m were common. Many of the rejected distances included ramps at complex roadway interchanges. In other cases, large differences can be explained by anomalies in the GIMS cartography (see Figure 5-11 and Figure 5-12) or by differences in interpretation of the location of an anchor point at the end of an anchor section (see Figure 5-13).

The rms difference of 18.6m exceeds the 2.1m accuracy requirement for anchor section distances. The GIMS cartography anchor section distances were also compared to the adjusted low-resolution orthophoto anchor section distances. The comparison statistics appear in the second row of Table 5-6. For rejected distances, differences as large as 100m were common. Once again, many of the rejected distances included ramps at complex interchanges. The rms difference of 13.8m exceeds the 2.1m accuracy requirement for anchor section distances. Therefore, GIMS cartography does not meet the accuracy requirement for survey of anchor section distances.

Table 5-6 GIMS Cartography Statistics

Reference Method	Object	Initial Sample	Iterations	Number Rejected	Mean	Std. Dev.	RMS
Videolog Van DMI	Anchor Section Distances	237 pairs	8	36(15%)	1.6m	18.6m	18.6m
Low-Resolution Orthos	Anchor Section Distances	218 pairs	8	29 (13%)	3.7m	13.4m	13.8m
Kinematic GPS Coords.	Anchor Point Coords.	72 pairs	3	4 (6%)	14.6m	7.2m	16.3m
Low-Resolution Orthos	Anchor Point Coords. Operator 2	438 pairs	7	73 (17%)	12.0m	6.4m	13.6m

The GIMS cartography anchor point coordinates were compared to the kinematic GPS anchor point coordinates. The comparison statistics appear in the third row of Table 5-6. Only four locations were rejected. The rms positional difference of 16.3m exceeds the 1.0m pointing error allowed for anchor point coordinates. The GIMS cartography anchor point coordinates were also compared to the low-resolution orthophoto anchor point coordinates of operator 2. The comparison statistics appear in the fourth row of Table 5-6. For rejected locations, positional differences as large as 60m were common. The rms positional difference of 13.6m exceeds the 1.0m accuracy requirement. Therefore, GIMS cartography does not meet the accuracy requirement for survey of anchor point coordinates.

Figure 5-11 Anchor Section 1108

Figure 5-12 Anchor Section 1085

Figure 5-13 Anchor Section 1051

5.2.8. Clean Cartography

Approach

After base records were tagged with anchor section identifiers and the cartography was cleaned by, for example, snapping together endpoints of strings, anchor section distances were derived from the clean cartography by first calculating string lengths and then summing them by anchor section. Anchor point coordinates were also extracted from the clean cartography. The procedures were executed with Geomedia Pro and MS Access.

Accuracy Criteria

The clean cartography does not meet the accuracy requirements for survey of the linear datum, for either anchor section distances or anchor point coordinates.

Other Criteria

This method uses GIMS cartography where divided roadways have been added and the cartography has been topologically structured. Therefore, the significant disadvantage to this method is the effort, staff skills, and independent source required to enhance the existing GIMS cartography. An independent source, such as orthophotos or local government cadastral digital databases, is necessary to create divided roadways and to modify the cartography to achieve appropriate accuracy values.

Achievement of appropriate accuracy levels includes creating more detailed line work in the cartography (for distance calculations) and finding more accuracy in absolute positioning of the cartography (for anchor point placement and distance calculations). Finally, most cartography is planimetric (does not include elevation). Therefore, the distances are foreshortened in areas with significant changes in elevation (Mississippi River Valley).

The cartography can represent the road types and segment types in Iowa except for roadways that converge and diverge (e.g., ramps and connectors). For converging and diverging roadways, the cartography does not contain the necessary information for anchor point placement. For roadways other than those that converge and diverge, the quality of the datum data depends on the resolution of the cartographic source. For example, 1:100,000 scale cartography has poor geometry for converging and diverging roadway intersections (ramps).

GIMS section maintenance creates undesirable breaks in the cartography. These breaks would increase the amount of interaction required for the user to interact with the cartography to create a datum design and perform datum collection. However, the impact is not severe.

Finally, the cartography is not produced when the datum is required; that is, the cartography does not meet the temporal requirements of the datum. Iowa DOT’s current processes create cartography significantly later than when roadways are first constructed (months or years). To meet these requirements would require significant changes in the way Iowa DOT interacts with local governments to acquire this information.

Findings

The clean cartography method is not desirable for initial datum creation or for operational update of the datum. The method does not produce data that meets the LRS accuracy requirements. To clean and position the cartography will require applying an independent source of higher accuracy. Finally, the current cartographic update processes would produce datum data significantly later than required. Having said all this, however, the cleaned cartography would be valuable as a source for datum design for initial datum creation.

Description of Accuracy Criteria

The clean cartography anchor section distances were compared to the adjusted videolog van DMI distances. The comparison statistics appear in the first row of Table 5-7. For rejected distances, differences as large as 100m were common. Many of the rejected distances included ramps at complex roadway interchanges. The rms difference of 17.1m exceeds the 2.1m accuracy requirement for anchor section distances. The clean cartography anchor section distances were also compared to the adjusted low-resolution orthophoto anchor section distances. The comparison statistics appear in the second row of Table 5-7. For rejected distances, differences as large as 100m were common. Once again, many of the rejected distances included ramps at complex interchanges. The rms difference of 14.1m exceeds the 2.1m accuracy requirement for anchor section distances. Therefore, the clean cartography does not meet the accuracy requirement for survey of anchor section distances.

Table 5-7 Clean Cartography Statistics

Reference Method	Object	Initial Sample	Iterations	Number Rejected	Mean	Std. Dev.	RMS
Videolog Van DMI	Anchor Section Distances	237 pairs	10	39(16%)	0.7m	17.1m	17.1m
Low-Resolution Orthos	Anchor Section Distances	218 pairs	9	28 (13%)	3.5m	13.7m	14.1m
Kinematic GPS Coords.	Anchor Point Coords.	72 pairs	3	4 (6%)	14.6m	7.2m	16.3m
Low-Resolution Orthos	Anchor Point Coords. Operator 2	437 pairs	8	77 (18%)	11.9m	6.1m	13.4m

The clean cartography anchor point coordinates were compared to the kinematic GPS anchor point coordinates. The comparison statistics appear in the third row of Table 5-7. Only four locations were rejected. The rms positional difference of 16.3m exceeds the 1.0m pointing error allowed for anchor point coordinates. The clean cartography anchor point coordinates were also compared to the low-resolution orthophoto anchor point coordinates of operator 2. The comparison statistics appear in the fourth row of Table 5-7. For rejected locations, positional differences as large as 60m were common. The rms positional difference of 13.4m exceeds the 1.0m accuracy requirement. Therefore, the clean cartography does not meet the accuracy requirement for survey of anchor point coordinates. Furthermore, comparison of Table 5-6 and Table 5-7 indicates no significant difference between the accuracies of the GIMS cartography and the clean cartography.

5.2.9. Project Plans

Approach

Six sets of project plans were used to reconstruct the alignment along U.S. 30 from just west of Nevada to the eastern end of the Field Pilot task area. The reconstruction included some anchor sections along ramps. The procedure employed the coordinate geometry functionality of Geomedia Pro. The Field Pilot Team chose GeoMedia Pro as opposed to a roadway design application (for example, CEAL) because it is targeted as the GIS interface for LRS maintenance and could potentially eliminate the need for LRS maintenance staff to learn and use multiple tools.

Since project plans are not inherently registered to any ground coordinate system, the placement of the initial point for the alignment and orientation of the alignment in azimuth were done by registration to the low-resolution orthophoto. Therefore, coordinates derived from project plans, for the purpose of the Pilot task, are not independent from the orthophotos and comparison with orthophoto coordinates should only be done with this in mind.

Accuracy Criteria

In general, project plans produced results that do not meet the accuracy requirements for survey of the linear datum for either anchor section distances or anchor point coordinates. The pilot team expected the plans to meet the accuracy requirements. The plans are not without ambiguity when attempting to compile datum objects based on the business rules applied in the Pilot. This is especially true for ramps. Because of the limited sample set for the pilot, these objects were not rejected and influenced the results. Another possible explanation is that the procedures and tools used to construct anchor section positions from the stationing reference line did not always produce the expected positions.

Other Criteria

This method probably uses the richest source of information for the datum. Unfortunately, project plans are difficult to acquire, especially digital data for Iowa DOT projects and digital or hardcopy plans for non-Iowa DOT projects. The amount of information and the lack of consistent representation of information over time make interpretation of plans for datum survey a significant challenge that requires a specialized skill.

Project plans can represent all surfaces and configuration fairly well, except for ramp configurations. There appears to be disagreement between where taper points for ramps are defined by DOT construction and the LRS design. The curvilinear nature of project plans is fairly good as well. This is less true in areas of significant elevation change, but can be improved if three-dimensional roadway design is implemented.

A significant disadvantage of this method is the timeliness of the information. As-built plans are typically compiled and made available to others after the roadway is completed. However, design plans can be considered as an alternative source since any modification to the alignment is sent back to Design.

Findings

More analysis is required to determine why the project plans did not meet the LRS positional accuracy requirements as expected. It is expected that more recent plans, especially those that are already digital, will reduce errors and anomalies and will meet the accuracy requirements.

Project plans are not desirable for initial datum creation because of the difficulty to produce large quantities of data with single workflow processes (a desired approach for initial data creation activities). Although project plans are expected to meet accuracy requirements, project plans are less desirable for datum maintenance due to their limited availability, the difficulty to extract datum data, and the current timeliness of the information.

Description of Accuracy Findings

Project plan anchor section distances were compared to the adjusted videolog van DMI anchor section distances. The comparison statistics appear in the first row of Table 5-8. The initial comparison included two anchor section distances for which the business rules for defining taper and gore points were different between the project plans and the Field Pilot task, causing exceptionally large distance differences even though they were within three standard deviations of the mean. After deletion of these two distances, the rms distance difference was 39.7m. Project Plan anchor section distances were also compared to the adjusted low-resolution orthophoto anchor section distances. The comparison statistics appear in the second row of Table 5-8. The rms distance difference is 46.3m. Therefore, project plans do not meet the accuracy requirements for survey of anchor section distances.

Table 5-8 Project Plans Statistics

Reference Method	Object	Initial Sample	Iterations	Number Rejected	Mean	Std. Dev.	RMS
Videolog Van DMI	Anchor Section Distances	8 pairs 6 pairs*	1	0(0%)	-44.2m -33.7m*	251m 20.9m*	239m 39.7m*
Low-Resolution Orthos	Anchor Section Distances	5 pairs	1	0 (0%)	-29.4m	40.0m	46.3m
Low-Resolution Orthos	Anchor Point Coords. Operator 2	15 pairs 12 pairs**	1	0 (0%)	45.1m 18.8m**	62.0m 21.8m**	75.0m 28.8m**

* After rejection of two large differences attributable to incompatibility between project plans and business rules used for anchor point selection on the Field Pilot task.

**After rejection of three large differences attributable to incompatibility between project plans and business rules used for anchor point selection on the Field Pilot task.

There were no matches between anchor points surveyed by kinematic GPS and anchor points computed from project plans. Therefore, the only coordinate comparison is with the low-resolution orthophotos, upon which the project plan coordinates are dependent. The statistics for this coordinate comparison appear in the third row of Table 5-8. The initial comparison included three anchor points with very large differences, even though those differences fell within three standard deviations of the mean. After rejection of these three anchor points, the rms positional difference was 28.8m, indicating that project plans do not meet the accuracy requirements for determination of anchor point coordinates.

5.2.10. Roadware Van DMI and DGPS

Approach

Five Roadware van DMI anchor section distances, five Roadware van DGPS anchor section distances, and ten Roadware van GPS anchor section coordinates were requested to be extracted by Roadware from their data. Only three anchor section distances each for DMI and DGPS could be extracted. One of the Roadware van DMI distances differed from the corresponding adjusted videolog van distance by –132m. This same distance, as measured by Roadware van DGPS differed from the adjusted videolog van distance by –137m.

Only seven anchor point coordinate pairs could be extracted from the Roadware van DGPS data. The rms positional difference between Roadware van DGPS anchor point coordinates and low-resolution orthophoto anchor point coordinates was 56.9m.

Accuracy Criteria

These results indicate that linear datum data from the Roadware van data is sparse. The numbers indicate that the Roadware van data do not meet the accuracy requirements for survey of the linear datum, but the sample sizes are too small to draw supportable conclusions.

Other Criteria

Basically, the technology and procedures used in the Roadware method combine DMI and DGPS based technologies. This field-based solution shares the same advantages and disadvantages described for the videolog van DMI and DGPS with a few exceptions. This method is contracted and would not require changing the priorities and practices of the current Iowa DOT videolog van. However, the current Roadware practices for IPMP data collection could not be used because the data collection requirements are different.

Findings

Not enough data was provided to make accuracy statements with any confidence. The current Roadware collection practices do not produce desirable results for datum collection or update.

5.3. Findings

No single method meets the positional accuracy requirements for anchor point placement, and anchor point span / anchor section distance measurement. Three methods had the best results: (1) Kinematic GPS meets accuracy requirements, but only for anchor point placement, (2) Videolog Van DMI meets the accuracy requirements, but only for anchor section distance measurement, and (3) low resolution orthophotos meets accuracy requirements for anchor section distance and anchor points, but only where anchor points are clearly photo-identifiable.

No single method produced the all-around best results for all method selection criteria. Some of the methods that met accuracy requirements are expensive. Methods that limited fieldwork could not be used to collect all roadways. Field methods could be used to collect all roadways but would be cost-prohibitive for initial data collection.

Consequently, a combination of methods will provide an acceptable solution for datum collection. In general, low-resolution orthophotos and DMI supplemented by DGPS/INS are the two most appropriate methods. If accuracy requirements are relaxed, existing data can be used in many circumstances but Iowa DOT will be required to perform data cleaning and improve data quality control / quality assurance procedures.

Whenever the Videolog Van DMI methodology is used, calibration runs should be made at both the beginning and end of the day and differences prorated over the data collection period, as was done on the Pilot task. Differences larger than 1m per mile between calibrations were detected during the Pilot task.

Project plans did not meet the LRS accuracy requirements as expected. The procedures used, sample size, and business rule differences for ramps help explain the rejection. Further analysis using more current data may produce more positive results.

Iowa DOT should consider investigation of other methods that involve orthoimagery. For example, there are low-cost photogrammetric methods for producing orthophotos from existing U.S.G.S. DEMs and National Aerial Photography Program photos. Another possibility is ortho-rectified high-resolution (1m) satellite imagery such as IKONOS.

The specifics for initial datum creation and datum maintenance are described below.

5.3.1. Initial Datum Creation

The best viable choice for both anchor sections and anchor points is orthophotos (high resolution and low resolution, respectively). Existing field inventory distances should be used in quality assurance tasks as an independent source, especially in geographic areas with significant elevation change. This method is best for all roadways except ramp-related roadways. Orthophotos and kinematic GPS could be used to collect ramp-related roadways (use kinematic GPS to locate the anchor point and use the orthophoto with the kinematic GPS results to create the anchor section), but this may prove to be more expensive than other methods.

The next viable choice is field DMI complemented by DGPS/INS technology. This method should be used where orthophotos are not available and for all ramp-related roadways. There are significant costs in independent field measurement (all roadways would be in the low millions of dollars). GIMS field inventory distances should be used in quality assurance tasks as an independent source. The field DMI/DGPS/INS should be used when confidence is low in orthophoto and existing field inventory data. The Videolog Van technology used in this Pilot should not be used as-is unless significant software enhancements are made.

Kinematic GPS is the most accurate method for anchor point coordinate collection. Given that the cost is extreme and safety concerns high, this method should only be used when the LRS accuracy requirements must be very reliable.

5.3.2. Datum Maintenance

All currently practiced methods that were analyzed do not have procedures, technology, or policies that allow the datum to be created for new or changed roadways prior to the roadways opening to traffic. Regardless of the method employed, Iowa DOT must make changes in order to meet the LRS temporal requirements at least on key roadway changes (as determined by LRS Board of Directors).

Roadway improvement project plans did not meet accuracy requirements, but are expected to if further analysis is performed. Project plans are expected to be the first choice for datum maintenance. However, these are limited in availability and are not viable unless the following constraints are met: 1) the source is digital and compatible with the LRS format: failing this a conversion program to change the source to a format compatible with the LRS is required, 2) the source must have a very accurate absolute coordinate system reference, and 3) the data is received prior to the road opening to traffic. Project plans cannot be used to collect ramp-related roadways without changes to roadway design and construction practices.

Orthophotos are the next appropriate option. They are, however, less likely to contain active roadway changes, especially prior to the roadway opening to traffic. Orthophotos can be used to collect ramp-related roadways, but must be augmented by a field method used to collect absolute locations of the converging or diverging roadway anchor point (e.g., kinematic GPS).

Of the methods tested, the field DMI/DGPS/INS combination method likely will be used most often for maintaining the datum. If project plans are not available, this combination is the most appropriate to collect all ramp-related roadways. The Videolog Van technology used in this Pilot should not be used.

Finally, Kinematic GPS is the most accurate method for anchor point positioning, but costs and safety concerns limit it as a practical alternative. When positions must have a high degree of accuracy or data validation problems occur this method can be used in a limited capacity.

5.3.3. Organizational Impacts

As outlined in the Physical Design Summary Document, the Office of Transportation Data should be responsible for surveying the datum. System Monitoring should be primarily responsible for most field-based data collection. The existing Iowa DOT field inventory collection processes managed by the Office of Transportation Data should include datum collection. These processes would need to incorporate the collection methods described above either directly or indirectly (contracted). The processes will need to be adjusted to meet the LRS temporal and positional accuracy requirements.

System Monitoring would use the DMI/GPS/INS and KGPS methods. The DMI/GPS/INS would require using the videolog van (if technology is upgraded and the van is available) and the need to incorporate GPS/INS technology into their current field inventory processes. The KGPS method would be needed, but this will occur infrequently. Therefore, the District Offices would need to assist with the physical collection of data using the KGPS method. The Office of Design and the Office of Construction will provide support roles. The District Offices will also be placing anchor point witness monuments when necessary.

System Management should be responsible for office-based data collection. Data collection methods include project plans, orthophotos, and the use of field inventory data for validation. For the project plans method, System Management will need assistance from the District Offices, the Office of Design, and the Office of Construction.

The LRS Manager should contract initial data collection. However, the Office of Transportation Data - as well as the District Offices, Office of Design, and Office of Construction - should have some staff participate to gain experience, validate data, and to make improvements in the processes and procedures for datum maintenance.

5.4. Practical Recommendations

5.4.1. Initial Datum Creation

The LRS Datum should be created by using the best orthophotos available. Existing field inventory distance measurements from GIMS will be used as an independent measure for quality control.

Accuracy assessments will need to be made for the USGS orthophotos since they were not assessed in the pilot. This will provide statewide coverage, but will probably require relaxing the accuracy requirements. These orthophotos are currently available for the entire state.

DMI/DGPS measurements will be used to fill in areas in the Datum where the relaxed accuracy requirements cannot be met by the orthophoto/field method. In order to make these measurements vehicles will need to be equipped with accurate DMIs and GPS units will need to be acquired. Software will also need to be written to collect and reduce the data once it has been collected. While it is expected that the anchor section length accuracy can be met with this method, the anchor point placement to within one meter cannot. However with the improvements in GPS receivers and the removal of SA (Selective Availability) accuracies approaching 2 meters are achievable using differential correction.

Ramp datum objects cannot be created using the orthophoto/field inventory method. These objects will be gathered by using a combination of orthophotos and GPS locations. Another possible method is to use DMI/DGPS to gather these points.

5.4.2. Datum Maintenance

Iowa DOT roadway improvement project plans will be used for datum maintenance on the primary road system

Obtaining plans from local governments, particularly in the larger urban areas will be investigated during the prototyping project.

Field inventory using the DMI/DGPS method will be used to verify the information obtained from the plans.

Field inventory using DMI/DGPS will be used to collect roadways when plans are not available. The currency of this collection will be decided during the prototyping project.

5.4.3. Organizational Impacts

Some increase in staff will be required for creation and maintaining the LRS.

The Office most impacted will be Transportation Data. They are the Office tasked with most Datum creation and maintenance activities. The System Monitoring Section will be responsible for most field activities. The System Management Section will be responsible for office based data collection processes.

District Office personnel will be involved in reference post, as built plan, and design plan activities.

Highway Division Staff will be involved in maintenance activities involving design plans.

6. PROCESS: ADJUST LINEAR DATUM

6.1. Process Requirements

The field pilot tests the methods chosen on how to perform the process requirement “Adjust Linear Datum.” This section provides a description of the elementary process, as well as the function and process under which it is defined.

Location Reference Maintenance (function)
Description: Ongoing activities necessary to maintain the LRS databases and the interfaces to Iowa DOT operational systems.

Conduct Linear Datum Survey (process)
Description: Carry out the fieldwork necessary to build the linear datum.

Survey Linear Datum (process)
Description: Establish field locations and distances for datum objects.

Adjust Linear Datum (elementary process)
Description: Determine the most likely length of the anchor sections by distributing the measurement errors.

6.2. Process Approach

Adjustment of the linear datum uses a mathematical method referred to as “least squares” to distribute the errors in the measurements and produce the best possible results for anchor section distances. The least squares method takes advantage of redundant measurements to come up with best estimates for errors and to determine the internal consistency of a data set for a given measurement method. It can be shown mathematically that the least squares approach produces the best possible results and, therefore, is the method of choice for computing anchor section distances whenever redundant measurements are involved. Redundant measurements are those that either duplicate others (e.g., double-measured anchor sections) or duplicate some combination of others (e.g., anchor point spans). One of the further advantages of least squares and redundant measurements is that blunders, or mistakes, in the measurements can be detected during analysis. Without redundancy, mistakes go undetected and contaminate the results.

There are a number of statistics associated with least squares adjustment that are used to control the adjustment process and to determine whether or not the results are acceptable. First of all, the input to a least squares adjustment includes not only the measurements but also estimated standard deviations in those measurements. These standard deviations are used to proportionately “weight” each measurement during the adjustment process. Measurements with small input standard deviations are likely to receive small corrections during the adjustment process, while measurements with large standard deviations are likely to receive large corrections.

Secondly, the least squares adjustment process includes computation of the “number of degrees of freedom” in the system of measurements. The number of degrees of freedom is equal to the number of redundant measurements, that is, the number of measurements beyond that which is necessary for computing each anchor section distance uniquely. A large number of degrees of freedom means that the errors in the measurements call be estimated well and that if blunders or mistakes are present they are likely to be detected.

Thirdly, the least squares adjustment process produces a quality control statistic called “the standard deviation of unit weight”. This number should be approximately equal to 1. If it is not, it indicates that something is amiss with the adjustment (e.g., blunders are present in the data). The standard deviation of unit weight is used as a flag to alert the analyst to problems with the data. Very often, a given data set will require a number of adjustments until an acceptable value for the standard deviation of unit weight is achieved.

Fourthly, the least squares adjustment process includes computation of the standard deviation in each adjusted anchor section distance. These standard deviations indicate the quality of each final value for anchor section distances. Taken together, their root mean square can be compared to the accuracy criterion of 2.1m in an anchor section distance to determine the acceptability of a measurement method for survey of the linear datum.

A more technical discussion of the least squares adjustment method appears in Appendix 11.7.

The Field Pilot task had three data sets that contained redundant measurements for the linear datum: 1) Videolog Van DMI, 2) Videolog Van GPS/INS, and 3) low-resolution orthophotos. Least squares adjustments were performed for each of these data sets. The remaining methods that were tested contained no redundant data and, therefore, did not lend themselves to least squares adjustment.

Software was written that implemented the least squares adjustment method. The same software was used to adjust the data sets from each of the three measurement methods. The software includes a rather sophisticated equation optimizer that should allow adjustment of very large data sets during full-scale implementation of the LRS.

6.3. Analysis of Findings

For each of the three measurement methods whose data sets were adjusted by least squares, the standard deviation of unit weight, the number of degrees of freedom, rms standard deviation in an anchor section distance, and the number of rejected measurements (blunders) are reported in the analysis below.

6.3.1. Videolog Van DMI

The Videolog Van DMI measurement method meets the desired accuracy criterion for anchor section distances if measurements made with the van in motion are excluded from the data set. Otherwise, this method does not meet the desired accuracy criterion. It is apparent videolog van measurements are associated with the nearest full second of measurement time (see Section 8.2.2) and that, therefore, measurements started or stopped with the van in motion are subject to error.

Summary statistics appear in Table 6-1. For measurements taken with the van stopped, the rms standard deviation in an anchor section distance of 2.3m is not statistically different from the accuracy criterion of 2.1m at a 4% level of significance. Rejected measurements included some at dead-ends and other peculiar field conditions where repeat positioning of the videolog van was difficult.

Table 6-1 Summary Statistics form Least Squares Adjustment of Videolog Van DMI Measurements

Included Data	Std. Dev. of Unit Weight	# Degrees of Freedom	RMS Std. Dev. in AS	# Rejects and % of Total
All Measurements	1.064	239	2.5m	24 (5%)
Measurements with Van Stopped	1.043	201	2.3m	79 (16%)

The final adjusted data set of 238 anchor section distances were used as a reference for comparison with other measurement methods.

6.3.2. Videolog Van GPS/INS

The videolog van GPS/INS method does not meet the accuracy requirement for survey of the linear datum. Summary statistics from the least squares adjustment appear in Table 6-2. The rms standard deviation in an anchor section distance of 3.9m is significantly larger than the accuracy criterion of 2.1m. The 59 rejected measurements included not only measurements where some difficulty was encountered in the field (as with the videolog van DMI measurements) but also measurements where spurious data were present (for example, see Figure 5-4, Figure 5-8, and Figure 5-6). Spurious data is caused when the software attempts to fill in data based upon the last known DGPS position by using INS data.

Table 6-2 Summary Statistics from Least Squares Adjustment of Videolog Van GPS/INS Measurements

Included Data	Std. Dev. of Unit Weight	# Degrees of Freedom	RMS Std. Dev. in AS	# Rejects and % of Total
All Measurements	1.088	244	3.9m	59 (11%)

6.3.3. Low-Resolution Orthophotos

The low-resolution orthophotos meet the accuracy requirement for survey of the linear datum. Summary statistics from the least squares adjustment appear in Table 6-3. The rms standard deviation of 2.3m in an anchor section distance is not significantly different form the accuracy criterion of 2.1m at a 4% level of significance. A least squares adjustment of these data was possible because all anchor section had been measured by each of two operators, thereby providing redundancy. Some anchor section spans measurements were also included. The 69 rejected measurements resulted, primarily, from operator interpretation of the locations of taper points, gore points, and termini of roadways, including incorrect decisions as to public versus private roadway and public roadway versus parking lot (for example, see Figure 5-2 and Figure 5-3).

Table 6-3 Summary Statistics from Least Squares Adjustment of Low-Resolution Orthophoto Measurements

Included Data	Std. Dev. of Unit Weight	# Degrees of Freedom	RMS Std. Dev. in AS	# Rejects and % of Total
All Measurements	1.066	205	2.3m	69 (14%)

The final adjusted data set of 218 anchor section distances served as a reference for comparison with other measurement methods.

6.4. Findings

The least squares adjustment method is a viable method for distributing error within the datum. It can be proven to yield the best possible results from the data at hand and should be used anytime redundant measurements are present

The software developed for the Pilot task includes an equation optimizer that will facilitate adjustment of very large data sets during full-scale implementation. The software is written in FORTRAN and requires a structured ASCII file for input. Input data files can be constructed with spreadsheets and text editors. The output from the program is an ASCII file containing a full report on the least squares adjustment. The software should be usable by Iowa DOT with a few minor modifications (e.g., for interactive input of standard deviation models which are now hard-coded). A tutorial and, perhaps, a brief training session on the application of least squares and use of the software will be necessary.

The least squares adjustment of the Field Pilot task data yielded results from both the videolog log van DMI and the low-resolution orthophotos that meet the accuracy requirements for survey of the linear datum. The number of rejected videolog van DMI measurements might have been reducible by accounting (e.g., with COGO computations) for irregular field conditions (e.g., dead-ends). The number of rejected orthophoto measurements might have been reduced by having better business rules in place to deal with special field situations (e.g., parking lots). The accuracy of the videolog van DGPS/INS measurements might have been improved by enhancements in the data reduction software. The current software computed coordinates for all positions even when no GPS data was received for extended periods of time.

The Office of Transportation Data, System Management should be responsible for performing these processes. There is no change from the Physical Design Summary Document. This Office should contract initial datum adjustment but have some Office staff participate to gain experience, validate data, and to make improvements in the processes and procedures for datum maintenance.

6.5. Practical Recommendations

Software will be developed and training will be provided for the least squares method. This software will be used for adjustment in both the datum creation and maintenance.

The Office of Transportation Data, System Management will be responsible for performing these processes.

7. PROCESS: PLACE REFERENCE POST

7.1. Process Requirements

The field pilot tests the viability of various methods on how to perform the process requirement "Place Reference Post". This section provides a description of this elementary process as well as the function and processes under which this elementary process is defined.

Location Reference Maintenance (function)
Description: Ongoing activities necessary to maintain the LRS databases and the interfaces to Iowa DOT operational systems.

> **Establish Linear Reference Method (process)**
> Description: Set up the reference objects used for location reference methods.
>
> > **Establish Reference Post (process)**
> > Description: Place, position, and publish fixed reference posts.
> >
> > > **Place Reference Post (elementary process)**
> > > Description: Locate and monument (put the post there) a reference post along a transportation route (this is a field activity). Measurement is from the datum reference object.

7.2. Process Approach and Analysis of Findings

Reference posts were located in the field using three different measurement methods: 1) kinematic GPS, 2) videolog van DMI, and 3) videolog van DGPS/INS. The kinematic GPS method produced coordinates for reference posts and the videolog van DMI and DGPS/INS produced offsets from anchor points. The kinematic GPS coordinates were used to compute offsets along anchor sections for comparison with the videolog van DMI method. The accuracy criterion for measurement of an anchor section offset to a reference post is 2.1m (see Appendix 11.1). The purpose of testing measurement methods for this process is to determine if the tested methods can be used to determine anchor section offsets to reference posts to within 2.1m. The Field Pilot only analyzed the positional accuracy of reference post placement. The additional criteria used to analyze the datum survey were not assessed for the reference post placement.

7.2.1. Kinematic GPS

The kinematic GPS coordinates for the reference posts were measured by placing the GPS antenna above the reference post and perpendicular to the posts base where it entered the ground. These coordinates are expected to be accurate to ± 1 decimeter or better.

For reference posts along long straight-aways, where anchor section offsets had been measured by videolog van DMI and/or DGPS/INS, coordinate inverses were computed to the appropriate anchor points, yielding straight-line distances between the anchor points and the reference posts. These straight-line distances then served as a basis for comparison with the anchor section offset measurements. Small differences arising from measurement to the posts by kinematic GPS, as opposed to along the shoulder for videolog van DMI and DGPS/INS, were deemed negligible.

7.2.2. Videolog Van DMI

Approach

The videolog van DMI offsets to reference posts were measured independently of one another. That is, the offset to each reference post was measured by returning to an anchor point to begin the measurement.

Accuracy Criteria

The videolog van DMI does not meet the accuracy requirement for locating reference posts along anchor sections. Summary statistics appear in Table 7-1. Although the sample is small (18), the differences between the videolog van DMI offsets and the offsets computed from the kinematic GPS coordinates are much larger than expected. Furthermore the large differences cannot be explained by the fact that DMI offset measurements are being compared to straight-line distances computed from coordinates. One would expect that the straight-line distances would be shorter, if anything. The differences reported in Table 7-1 are the computed minus the measured, and the mean difference is a positive 30.0m.

Table 7-1 Summary Statistics for Comparison of Kinematic GPS and Videolog Van DMI for Reference Post Locations

Number of Offsets	Mean Difference	Stand. Dev. of the Differences	RMS Difference
18	30.0m	16.2m	34.1m

The offset measurements had been collected "on-the-fly" with the van moving at something less than highway speed. Data are recorded once every second on board the van. It is possible that the end of a measurement is associated with the last data point (last full second of time) before the measurement is ended by the operator. Further evidence is provided by an experiment conducted by making ten "on-the-fly" measurements of the DMI calibration baseline at each of 20, 30, 40 and 50mph speeds. Each of these measurement sets produced average distances that were short of the known baseline length. Table 7-2 contains the difference between the known and average measured distance for each of the four speeds. It also contains the distance that the van travels in one second (the data recording interval). In all but one instance, the distance the van travels in one second is greater than the shortness of the average measurements. The discrepancy at 40mph could be attributable to operator uncertainty in marking the end of a measurement.

Table 7-2 Measurement Difference and Distance Traveled Versus Van Speed

Van Speed	20mph	30mph	40mph	50mph
Known Distance Minus Ave. Meas. Dist	7m	11m	22m	20m
Distance Van Travels in One Second	9m	13m	18m	22m

Findings

The Videolog Van DMI does not produce acceptable distances if the van does not come to a complete stop at the post. However, it is expected that the method will produce acceptable distances if the van does come to a complete stop at the post.

7.2.3. Videolog Van DGPS/INS

Approach

The videolog van DGPS/INS measurements were made at the same time as the videolog van DMI measurements. The reference posts offsets were measured independently of one another. That is, the offset to each reference post was measured by returning to an anchor point to begin the measurement.

Accuracy Criteria

The videolog van DGPS/INS method does not meet the accuracy requirement for measurement of reference post offsets. Summary statistics appear in Table 7-3. The small sample data set of 17 for comparison to kinematic GPS coordinate inverses yielded an rms difference of 49.6m. The videolog van DGPS/INS reference post offsets were also compared to the videolog van DMI reference post offsets. This resulted in a larger data set of 34 with an rms difference of 21.3m.

Table 7-3 Summary Statistics for Comparison of Videolog Van GPS/INS Reference Post Offsets

Reference Data Set	Number of Offsets	Mean Difference	Stand. Dev. of the Differences	RMS Difference
Kinematic GPS	17	37.1m	33.0m	49.6m
Videolog Van DMI	34	0.5m	21.3m	21.3m

Conclusions

The Video van DGPS/INS method is not a viable method. The required accuracy was not met and the technology produced spurious data. Other DGPS/INS solutions may not produce such spurious data.

7.3. Findings

The undesirable test results can be explained by the fact that videolog van DMI and DGPS/INS offset measurements to reference posts were collected “on-the-fly” with the van moving at something less than highway speed. It is, therefore, recommended that reference offset measurements be made in the future with the van stopped at each reference post and anchor point, except where safety may be an overriding concern. In the cases where the vehicle cannot be brought safely to a stop, field notes should indicate this fact and the accuracies of the associated reference post offsets should be expected to be lower than the design standard (2.1m).

However, there is no reason to expect that offsets to reference posts cannot be measured with the same accuracy if the vehicle is brought to a stop at both ends of a given measurement. Therefore among the tested methods, the Videolog van DMI is the method of choice for placing reference posts along anchor sections. Anchor section distances can be determined to within the acceptable tolerance with this measurement method.

The procedure of returning to an anchor point to begin each measurement to a reference post provides independence among the measurements, but in practice is probably too time consuming and costly. It is recommended, that further analysis be done to determine if cumulative offset measurements can be made to successive reference posts without returning to an anchor point to begin each measurement. Substantial savings in time and effort should be realized if this approach could be implemented. An appropriate procedure for flagging data measured this way (as apposed to each post from the anchor point) will need to be developed.

It is unclear who is responsible for performing these processes (see the Physical Design Summary Document). PMIS Support, the Districts, and the Office of Maintenance are all involved in reference post management. However, the methods tested are currently not used to locate reference posts. The Office of Transportation Data, System Monitoring has the expertise to collect reference post locations using the Videolog Van DMI method. If the Office of Transportation Data uses the vehicle to collect the data, one of the aforementioned groups would post process it and enter into the LRS (PMIS Support does the inventory and would be the likely choice under current circumstances).

The Field Inventory method (DMI/DGPS-based) should be evaluated for reference post placement. The technology is much more affordable and accessible than the videolog van. Addition of DGPS to assist in data quality control is required. PMIS Support, the Districts, or the Office of Maintenance could acquire such technology and training.

As kinematic GPS becomes more economical and easier to use, this may be a viable method for locating reference posts. This method does assume the use of the coordinate route LRM and a cartography that has appropriate accuracies so accumulated error is still within LRS accuracy requirements.

7.4. Practical Recommendations

DMI with DGPS should be used to collect initial datum locations of the reference posts. If this proves to be too expensive, the new video log van will be used.

All reference posts should be shown on design plans.

PMIS Support should continue to coordinate reference post maintenance activities.

The LRS manager should contract out the initial reference post datum collection task. PMIS Support will be involved in this process so that they can improve the maintenance of the reference post locations.

8. PROCESS: PLACE STATION POST

8.1. Process Requirements

The field pilot tests the viability of various methods on how to perform the elementary process requirement "Place Station Post". This section provides a description of this elementary process as well as the function and process under which this elementary process is defined. In the pilot, station posts (posts or stamps) were measured from the beginning of the anchor section as opposed to the beginning of the project section as stated below. This strategy is what would be necessary to locate existing station posts.

Location Reference Maintenance (function)
Description: Ongoing activities necessary to maintain the LRS databases and the interfaces to Iowa DOT operational systems.

> **Establish Linear Reference Method (process)**
> Description: Set up the reference objects used for location reference methods.
>
> > **Establish Station Post (process)**
> > Description: Place, position, and publish fixed station posts or pavement stamps.
> >
> > > **Place Station Post (elementary process)**
> > > Description: Locate and monument (actually put the post or stamp) a station post or pavement stamp along a highway improvement project section. Measurement is according to project stationing.

8.2. Process Approach and Analysis of Findings

Station posts and station marks, by definition are expected to be placed at even stations or increments of multiples of 500 feet. A number of existing station posts and station marks were included in the Field Pilot task to test how well they had been placed with respect to one another. Four measurement methods were tested: 1) kinematic GPS, 2) videolog van DMI, 3) videolog van GPS/INS, and 4) project plans.

Two tests were performed for station marker placement:

1. Determine if station posts and stamps are spaced according to the stated stationing and
2. Determine which measurement methods can best be used to measure the relative spacing between station posts or stamps to within the desired accuracy tolerance of 2.1m. If this accuracy can be obtained between two stamps or posts than it is expected that the same accuracy relative to an anchor point can be obtained. If the method meets the accuracy requirements, it precludes having to measure from the anchor point to each post or stamp.

The Field Pilot only analyzed the positional accuracy of station marker placement. The additional criteria used to analyze the datum survey were not assessed for the station marker placement.

8.2.1. Kinematic GPS

Approach

Kinematic GPS coordinates were obtained for four sets of station posts, two sets of 8 posts in the eastbound and westbound directions of U.S. 30. Coordinate inverses were computed between each successive pair of station posts and the Project scale and elevation factors were applied, yielding straight-line on-the-ground distances between the pairs of posts. Each of the four sets of station posts was along a straight-away, so that straight-line distances computed from coordinates could be legitimately compared to stationing. The distances were then compared to the corresponding theoretical distances determined from stationing.

Kinematic GPS coordinates were also obtained on the same four sets of station stamps along U.S. 30. Coordinate inverses were computed between each successive pair of station stamps and the Project scale and elevation factors were applied, yielding straight-line on-the-ground distances between the pairs of stamps. The distances were then compared to the corresponding theoretical distances determined from stationing.

Accuracy Results

Station posts and stamps are not placed according to their stationing based on the LRS required accuracies (answer for Test 1). The comparison results appear in Table 8-1. The RMS difference of 6.0m indicates that these station posts are not placed exactly according to their stationing. The RMS difference of 7.1m indicates that these station stamps are not placed exactly according to their stationing. However, the similarity of the statistics for station stamps and station posts indicates consistency between their placement in the field.

8.2.2. Videolog Van DMI

Approach

Videolog van DMI offsets were measured to the four sets of station stamps. For each of the station stamp data sets, the measured offsets were reduced to relative offsets by subtracting the offset of the first station stamp in the set. The kinematic GPS coordinates were used to compute distances from the first station stamp in each set to all other station stamps in the set. The Project scale factor and the elevation factor were applied to these computed distances to yield on-the-ground straight-line distances. The measured relative offsets were then subtracted from the computed relative offsets.

Accuracy Results

The results are summarized in Table 8-1. The rms difference of 5.6m indicates that the videolog van DMI cannot be used to measure relative distances between station stamps (accuracy tolerance is 2.1m). This was not expected.

There is no reason to expect this number to be larger than the rms difference of 2.3m in an anchor section distance, reported in Section 5.2.2. The vehicle was brought to a stop at each station stamp. However, the sample size is relatively small (27). These facts suggest that the results are inclusive and that further testing is warranted to determine the best method for locating station stamps. On the other hand, it is clear that merely measuring an offset to one station stamp and then computing offsets to others by taking differences in stationing will not suffice to locate station stamps within the desired level of accuracy (i.e., 2.1m).

8.2.3. Videolog Van DGPS/INS

Approach

Videolog van DGPS/INS measurements of offsets between station stamps were obtained at the same time as the videolog van DMI measurements. For each of the station stamp data sets, the measured offsets were reduced to relative offsets by subtracting the offset of the first station stamp in the set. The kinematic GPS coordinates were used to compute distances from the first station stamp in each set to all other station stamps in the set. The Project scale factor and the elevation factor were applied to these computed distances to yield on-the-ground straight-line distances. The measured relative offsets were then subtracted from the computed relative offsets.

Accuracy Results

The results are summarized in Table 8-1. The videolog van DGPS/INS cannot be used to measure relative distances between station stamps (accuracy tolerance is 2.1m). The rms difference of 19.7m indicates that spurious data are probably present in the videolog van DGPS/INS measurements.

8.2.4. Project Plans

Approach

The six sets of project plans that were used to reconstruct the alignment along U.S. 30 were used to compute coordinates for 31 station stamps. As described in Section 5.2.9, coordinates of the reconstructed alignment depend upon orientation of the project plans to the low-resolution orthophoto. Coordinates for the station stamps were computed from the project plans assuming that the station stamps conformed with the theoretical station values. The computed station stamp coordinates were then compared with the kinematic GPS coordinates for the same station stamps.

Accuracy Results

The comparison statistics appear in Table 8-1. The project plans cannot be used to measure relative distances between station stamps (accuracy tolerance is 2.1m). The rms positional difference 14.3m includes not only the true differences between theoretical stationing and actual distances (see Section 8.2.1), but also errors associated with orientation of the project plan alignment with the orthophoto. It is also likely that the method used to collect the project alignments introduce additional error (see Section 5.2.9)

Table 8-1 Summary Statistics for Comparison of Kinematic GPS Coordinates for Station Markers to Other Methods

Method	Number of Differences	Mean Difference	Stand. Dev. of the Differences	RMS Difference
KGPS-Posts	27	0.3m	6.0m	6.0m
KGPS- Stamps	57	-1.9m	6.8m	7.1m
Videolog Van DMI – Stamps	27	-2.5m	5.0m	5.6m
Videolog Van GPS/INS - Stamps	27	-15.2m	12.5m	19.7m
Project Plans - Stamps	31	9.3m	11.0m	14.3m

8.3. Findings

Station posts and station stamps are not spaced according to theoretical distances derived from differencing station values. Unless further testing shows more reliable locations for station stamps and posts, they should be treated similarly to other LRMs such as reference posts. That is, field measurements as offsets to station markers should be treated differently from offsets computed from plans. This also helps explain why the project plan method failed to be a viable method for measuring relative distances between field station markers.

These results are disappointing because the costs to place all existing station markers (posts and stamps) relative to the datum (an offset from an anchor point) will be quite expensive. If the markers were spaced according to the theoretical distances, only the first station marker offset, per construction project, would need to be measured.

The videolog van DMI's failed to meet the accuracy requirement for measuring relative distances between station markers. These results are surprising. The videolog van DMI results for the datum survey suggest that the results here are inclusive and that further testing is warranted. On the other hand, merely measuring an offset to one station stamp and then computing offsets to another by taking differences in stationing will not suffice to locate station stamps within the desired level of accuracy.

The District Offices are responsible for performing these processes (see the Physical Design Summary Document).

The Field Inventory method (DMI/DGPS-based) should be evaluated for station marker placement. The technology is much more affordable and accessible than the videolog van. While an absolute location (x,y) is not required, it would be valuable to augment the DMI method with DGPS/INS to assist in data quality control. The Districts could acquire such technology and training.

As kinematic GPS becomes more economical and easier to use, this may be a viable method for locating station markers. This method does assume the use of the coordinate route LRM and a cartography that has appropriate accuracies so accumulated error is still within LRS accuracy requirements.

8.4. Practical Recommendations

The Stationing LRM will be automatically derived from design plans for new projects on the primary road system.

The Stationing LRM has been formally identified as having several design issues which will be described in this Project’s redesign document and will be dealt with in the prototyping project.

The Office of Design, with assistance from the District Offices, will provide the design plans to be used for the stationing process.

The Office of Design, with assistance from the District Offices, will create and manage the Stationing LRM data and processes.

9. Results of Hypotheses Testing

The primary pilot benchmarks, as summarized below, were all performed for the Pilot (Section 3.4). These results reflect the findings described in Sections 4 through 8.

Hypothesis 1: The accuracy requirements for development of the linear datum are high enough that changes in environmental and/or instrumental conditions, during a day's run with a DMI / DGPS vehicle, will require calibration of the instruments at both the beginning and end of each day.

Yes. Calibration is necessary in order to get the accuracies required. The DMI was calibrated at the beginning and end of each day’s use and differences were found (no statistical test was performed to determine significance). The calibration difference was prorated over the period of use.

Hypothesis 2: There is no significant difference between distances measured along centerlines of bi-directional roadways and centerlines of driven lanes on those roadways. That is, the business rules for anchor section representation and anchor section measurement do not conflict. "No significant difference" means a value less than or equal to a tolerance based upon the datum's design criteria for accuracy.

Yes. The root mean square of the residuals in the least squares adjustment of the videolog van DMI measurements was 2.3m when only those measurements taken with the van at a complete stop were included in the adjustment. The design criterion for the datum was 2.1m. There is no difference between 2.3m and 2.1m at a 4% level of significance for the 204 included measurements. The 2.3m statistic could also be interpreted as an estimate for the difference between distances driven along the centerlines of lanes and the centerline of the road for two lane roads.

Hypothesis 3: GPS coordinates, captured at rates within the capacity of the instrumentation and within feasibility for data management, represent the curvilinear nature of the roadway sufficiently to meet the accuracy requirements for measurement of anchor section distances.

No, but the data contained many anomalies that should be avoidable with differential GPS technology, especially since selective availability is no longer in force (it was still in force at the time of the data collection).

Hypothesis 4: GPS coordinates, captured on-the-fly with a moving vehicle, meet the accuracy requirements for anchor point coordinates.

No. The end of a measurement is associated with the last full second of time in the data stream. See Section 5.2.4 Videolog Van DGPS/INS.

Hypothesis 5: In flat areas, such as Story County, anchor section distances, measured by heads-up digitizing of high-resolution orthophotos, meet the accuracy requirements of the linear datum.

> Yes. The root mean square standard deviation in the anchor section distances, resulting from the least squares adjustment, was 2.3m The design criterion was 2.1m. There is no statistical difference between these two numbers at a 4% level of significance with the degrees of freedom of the data set.

Hypothesis 6: Historical versions of the datum can be developed from plans and then related to the current roadway system.

> Yes. This can be done in some cases. However, the project plans must be oriented by using an external reference (e.g., orthophotos) or must have projection / datum information established within the digital files.

Hypothesis 7: Existing Iowa DOT data collection processes (i.e., Roadware Van GPS and DMI, field inventory) can be leveraged to include data collection for the LRS (datum and reference objects).

> Inconclusive for the Roadware van data. Some of the requested data could not be generated.
>
> No, for the field inventory data unless accuracy requirements are relaxed.

Hypothesis 8: The business rules, described in Appendix 11.2 for anchor point selection and identification can be implemented in the field.

> Yes. The business rules can be implemented in the field. This was accomplished during the Field Pilot task. However, more rules are required for special circumstances (e.g., parking lots).

10. FUTURE CONSIDERATIONS

The Roadware method, or a similar provider, should be benchmarked again for datum survey. While the existing data and procedures from pavement rating collection were not viable, this approach to datum survey could prove to be quite viable for initial datum collection.

There are other datum collection methods that exist, but were not tested. Iowa DOT should consider analyzing such methods. First and most important is DGPS technologies with selective availability turned off. This should reduce costs and supply a more accurate product. In addition, low cost orthophoto production is possible if existing products are used (for example, using NAPP photography and existing digital elevation models). In the future, Iowa DOT may be able to rely more on using orthophotos for maintenance. Current research, such as TR-446 Iowa Highway Research Board, which is evaluating the feasibility of using soft photogrammetry to measure features along the roadway, should continue to be monitored.

Orthophotos from the USGS Digital Orthophoto Quarter Quad Program might prove valuable in areas where local governments in the state of Iowa do not have their own coverage. However, the USGS orthophotos may be as old as 10 years in some areas, and may not be as accurate as existing local government orthophotos. Finally, ortho-rectified high-resolution space imagery is another consideration. The number of value-added products is growing, resolution is increasing, and prices are becoming more competitive with other remotely sensed methods.

The Intelligent Transportation System industry is evaluating the use of telematics for locating vehicles (e.g., cellular phones triangulated between different cellular towers). As this technology stabilizes this may become a viable option much like GPS.

Iowa DOT should also consider evaluating whether the methods are viable for placing Literal Description Reference Features. Examples of such features are bridge expansion joints or abutments, and railroad crossings. Although the pilot team did not perform any formal analyses, an extrapolation of the pilot results suggests considerable value from pursuing these analyses.

There are considerably more reference features than datum objects or reference posts. Therefore, remotely sensed methods will most likely be significantly more economical than field methods. Orthophotos may prove viable for the initial creation of these reference features. High resolution may be required for bridge and railroad features. Use of the orthophotos in this context assumes that the datum itself is created from the same orthophotos. A Field DMI would most likely be the next favorable method. DGPS/INS combined with the DMI would add value and quality control but are not required.

For maintenance, the best method would most likely be the roadway improvement project plans. As described under Datum Maintenance, plans are only available in limited areas and must meet particular constraints to be a viable option. Timely orthophotos also may be difficult to find. The field DMI / DGPS method can most likely be used for all linear reference features. kinematic GPS can be used when accuracy requirements must be met, but costs and safety make it impractical for general use.

If LRS positional accuracy requirements are relaxed, the following methods can be explored. None of these methods were analyzed as part of the pilot. Existing data from the new videolog van can be explored to place reference posts along the datum. Intersections used as literal description reference features could come from GIMS but the same preprocessing requirements mentioned under survey datum hold here as well. The current positions of bridge and railroad crossings cannot be used because they represent locations other than those defined by current LRS business rules, however these locations could be modified so that they are useable by the LRS. Station marker positions can be interpolated along the datum, but positional accuracies are not known at this time.

11. APPENDICES

11.1. Derivation of Maximum Allowable Standard Deviation in an Anchor Section Distance

The following reasoning is based upon 1) the requirement for locating a reference post to within 3-5 meters along an anchor section with 90 percent certainty and 2) analysis of the repeated measurements of the calibration baseline and selected roadway segments conducted in August and September, 1999.

1. Assumption: Use the high end (5m) of the allowable range.

2. Compute the standard deviation in locating a reference post along an anchor section:

$$1.65\sigma_{RP} = \pm 5m \qquad \textbf{or} \qquad \sigma_{RP} = \pm 3.03m \tag{1}$$

where 1.65 is the multiplier for 90% certainty (1σ = 68% certainty) and σ_{RP} is the standard deviation in a reference post location along an anchor section.

3. If a reference post is located by measuring from the “from” anchor point of the anchor section, then σ_{RP} is merely equal to the standard deviation in that measurement. This provides no criterion for the required accuracy of the anchor section distance.

4. If the reference post is located by measuring from the “to” anchor point of the anchor section then, the offset for the reference post is obtained by subtracting the measurement from the anchor section distance:

$$RP = AS - RPM \tag{2}$$

where RP is the reference post offset along the anchor section, AS is the anchor section distance, and RPM is the measurement from the “to” anchor point to the post. This provides a criterion for the required accuracy of the anchor section distance.

5. Assumption: Equal uncertainty should be attributed to the anchor section distance and the measurement that locates the reference post. That is,

$$\sigma_{AS} = \sigma_{RPM} \tag{3}$$

where σ_{AS} is the standard deviation in the anchor section distance and σ_{RPM} is the standard deviation in the measurement to the reference post.

6. By the law of propagation of random error applied to equation (2),

$$\sigma_{RP}^2 = \sigma_{AS}^2 + \sigma_{RPM}^2 \qquad (4)$$

as long as AS and RPM are independent.

7. Substituting equation (3) into equation (4) and rearranging yields

$$\sigma_{AS}^2 = \frac{\sigma_{RP}^2}{2} \qquad (5)$$

8. Substituting equation (1) into equation (5) and solving for σ_{AS} yields

$$\sigma_{AS} = \frac{\pm 3.03}{\sqrt{2}} = \pm 2.14m \qquad (6)$$

the maximum allowable standard deviation in an anchor section distance.

9. Now, determine the maximum distance that can be measured for a single anchor section without including a spanning measurement for redundancy. Let the standard deviation in this distance be σ_{M1}. If the standard deviation in positioning the DMI at each end of the anchor section is ±1m, then

$$\sigma_{M1}^2 = 2.14^2 - 1^2 - 1^2 = 2.58m^2 \quad \textbf{or} \quad \sigma_{M1} = \pm 1.61m = \pm 5.3\,ft \qquad (7)$$

10. The manufacturer's stated accuracy of the DMI is ±1 ft per mile. Assumption: this means that $\sigma_{DMI} = \pm 1\,ft$ for a one-mile distance, ±2 ft for a two-mile distance, and so on (increasing linearly with distance).

11. Then the maximum number of miles (M1) that the DMI can measure without a redundant spanning measurement is 5.3 ft / 1 ft per mile = 5.3 miles = M1.

12. Using the methodology developed by Vonderohe and Hepworth (1998) for first and second order design, it was determined that the maximum anchor section distance for two adjacent anchor sections whose distances are measured <u>and</u> whose span is measured is six miles. This analysis also indicated that two adjacent anchor sections with seven-mile distances will have errors in their distance measures that cannot be compensated for by a redundant spanning distance. Thus, for the DMI technology and the uncertainty in positioning the van (as indicated from earlier experiments), six miles is a "magic" number. If we go less, we need no spanning measurement. If we go further, we cannot make up the error with a spanning measurement. Given the long, narrow nature of the Field Pilot task area, there are only a few distance measures spanning two anchor sections required for optimal design.

11.2. Datum and LRM Location Business Rules

This appendix describes how to locate anchor points to be observed for the eight cases that will be encountered in the field.

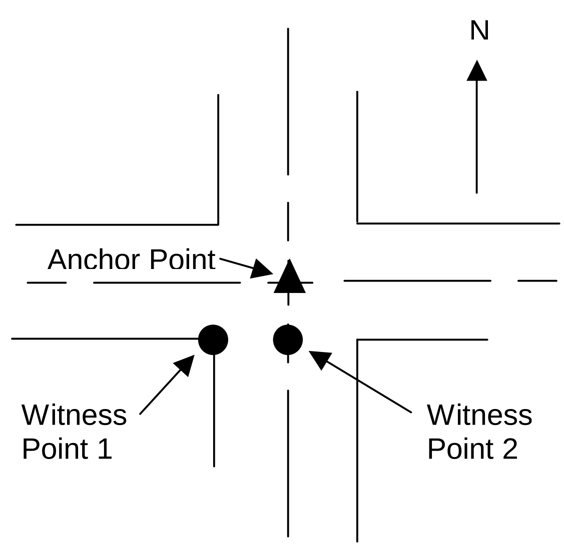
For all cases, a ***traveled way*** is defined as the section of the roadway on which vehicles travel. The traveled way does not include the shoulders or curbs. A ***pavement edge*** is defined as the edge of the roadway that has a clear, distinct end in bituminous or concrete surface, which may include the roadway shoulders

11.2.1. Cross Intersections

Business Rule: At cross intersections, anchor points are at the intersection of the centerlines of the traveled ways. See Figure 11-1. This rule holds for all surface types (e.g., concrete, bituminous, gravel, dirt, combination). In the instances where traffic precluded RTK measurements being taken at the anchor point, witness points are shown on the drawings and the business rule used is cited (see the Figure as well):

1. Witness point 1 uses the South / West rule and is used for RTK when both streets traffic preclude safe measurements in the roadway.
2. Witness point 2 uses the South / West rule and is used for RTK when only one streets traffic precludes safe measurements in the roadway.
3. In roadways with medians where the traffic precluded safe measurements an RTK measurement was taken on one of the gore noses following the South / West rule.

Figure 11-1 Anchor Points at Cross Intersections

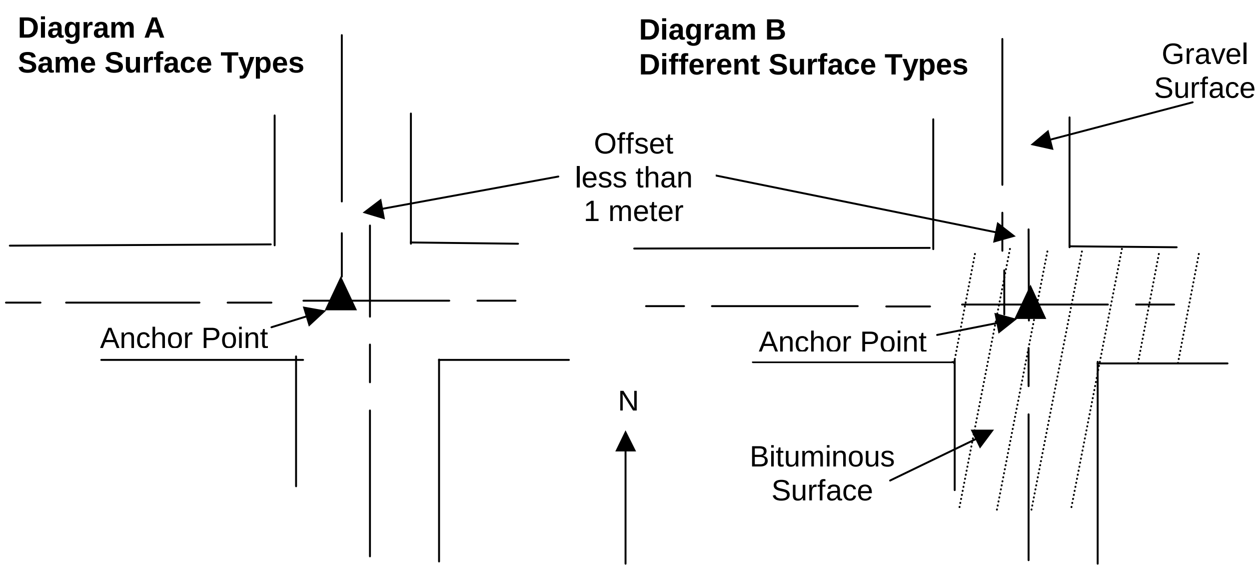


For roads with the same surface type when the crossing roads are offset, the South / West rule will apply. See Figure 11-2, Diagram A. That is, if the offset is primarily east-west, the anchor point will be located at the westernmost centerline intersection. If the offset is primarily north-south, the anchor point will be located at the southernmost centerline intersection.

For roads with offsets that have paved surfaces on two of the perpendicular legs, the centerlines of the perpendicular paved roads will be used (Diagram B).

For the purposes of measurement, anchor point location is to be determined visually by observing the edges of the traveled way in all directions and estimating the intersection of the centerlines. Distance measurements should begin / end at the anchor point. Coordinates of the anchor point should be determined.

Figure 11-2 Anchor Points at Cross Intersections that are Offset

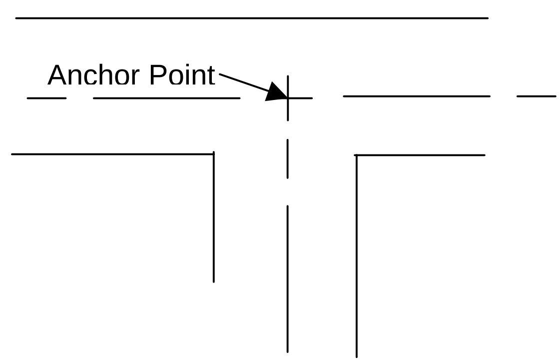


11.2.2. T Intersections

Business Rule: At T intersections, anchor points are at the intersection of the centerlines of the traveled ways (Figure 11-3). This rule holds true for all surface types (e.g., concrete, bituminous, gravel, dirt, combination). For roads with paved surfaces on two or more of the legs (one of which is perpendicular), the centerlines of the paved surfaces will determine the anchor point location.

For the purposes of measurement, anchor point location is to be determined visually by observing the edges of the traveled way in all directions and estimating the intersection of the centerlines. Distance measurements should begin / end at the anchor point. Coordinates of the anchor point should be determined.

Figure 11-3 Anchor Points at T-Intersections

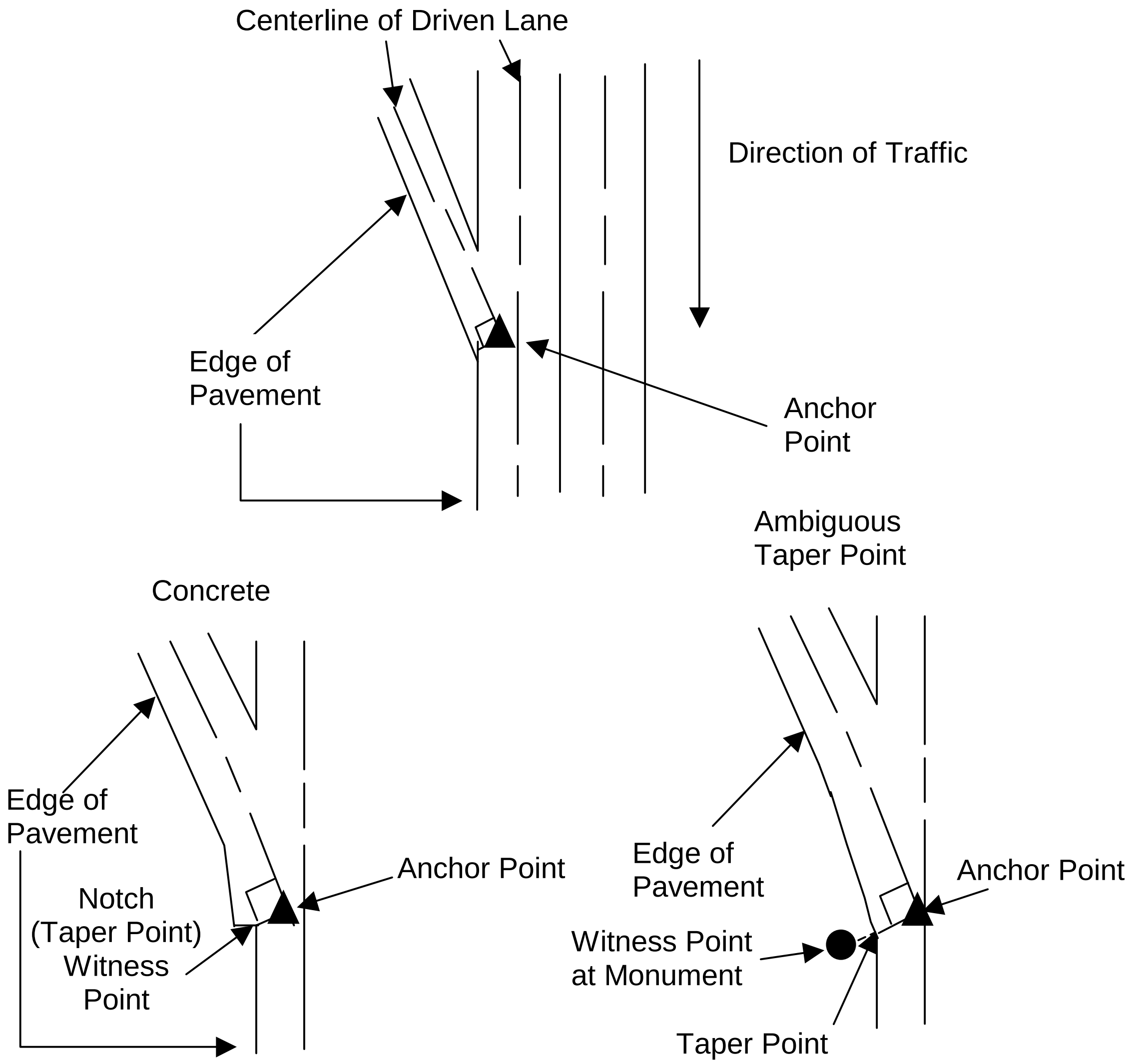


11.2.3. On and Off Ramps

Business Rule: At on and off ramps, anchor points are along the centerlines of driven lanes at perpendicular offsets from taper points (See Figure 11-4). Taper points are defined as intersections or transition points at outside pavement edges. Where taper points are ambiguous in the field (e.g., bituminous surfaces), they will be monumented by posts placed at the same distance from pavement edges as delineators. Anchor points defined in this manner approximate the intersection of the centerlines of the ramp and the right-most through driven lane.

For the purposes of measurement, the taper point should be identified and the location of the anchor point estimated from the location of the taper point. Distance measurements should begin / end at the anchor point. Coordinates of the anchor point should be determined. In some cases, the coordinates of the taper point will have been determined by carrier phase kinematic GPS. It should be noted that all of the ramp drawings are exaggerated and the actual difference between the angle in the drawing and a perpendicular to the centerline of the driving lane is very small.

Figure 11-4 Anchor Points at On- and Off-Ramps

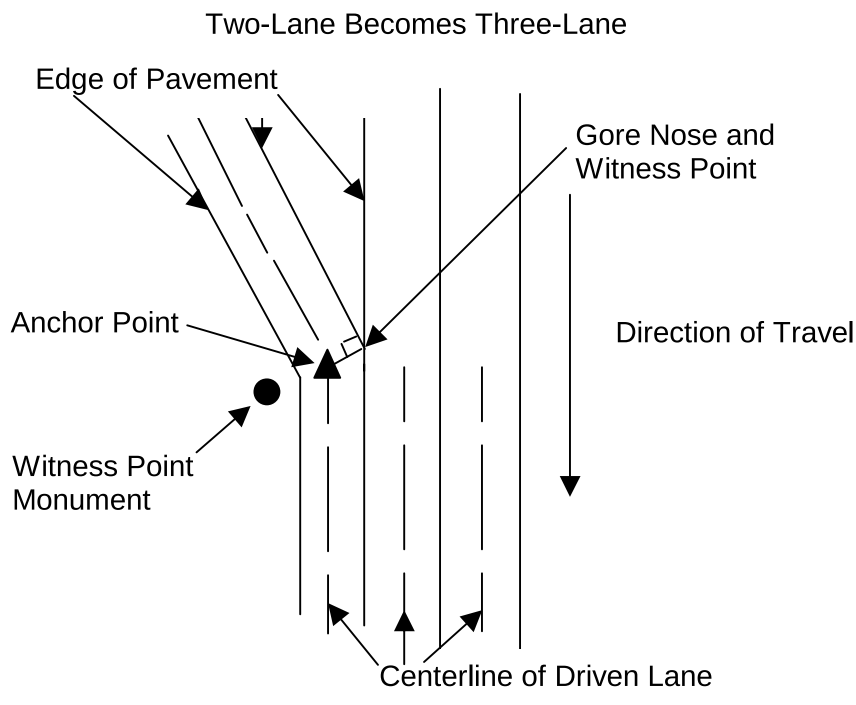


11.2.4. Ramp-Becomes-Lane / Lane-Becomes Ramp

Business Rule: Where a ramp becomes a lane or a lane becomes a ramp, the anchor point is on the centerline of the driven lane at a perpendicular offset from the gore nose (see Figure 11-5). The gore nose is defined as the intersection of the pavement edges. Some gore noses might be squared off or rounded. This situation applies not only in urban areas where the added lane continues to the next ramp, but also at cloverleaves where the added lane exists only at the interchange. At a cloverleaf an on-ramp becomes a lane and then becomes off-ramp. This circumstance calls for two anchor points, each witnessed by gore noses. In the cases where the gore location is ambiguous, monuments will be required. Please note that these angles are also exaggerated and that the difference from a perpendicular to the centerline of the drive lane is small.

For the purposes of measurement, the gore nose should be identified and the anchor point location should be estimated from the gore nose location. Distance measurements should begin / end at the anchor point. Coordinates of the anchor point should be determined. In some cases, the coordinates of the gore nose will have been determined by carrier phase kinematic GPS.

Figure 11-5 Anchor Points at Ramp/Lane Combinations



11.2.5. Two-Way Becomes Divided

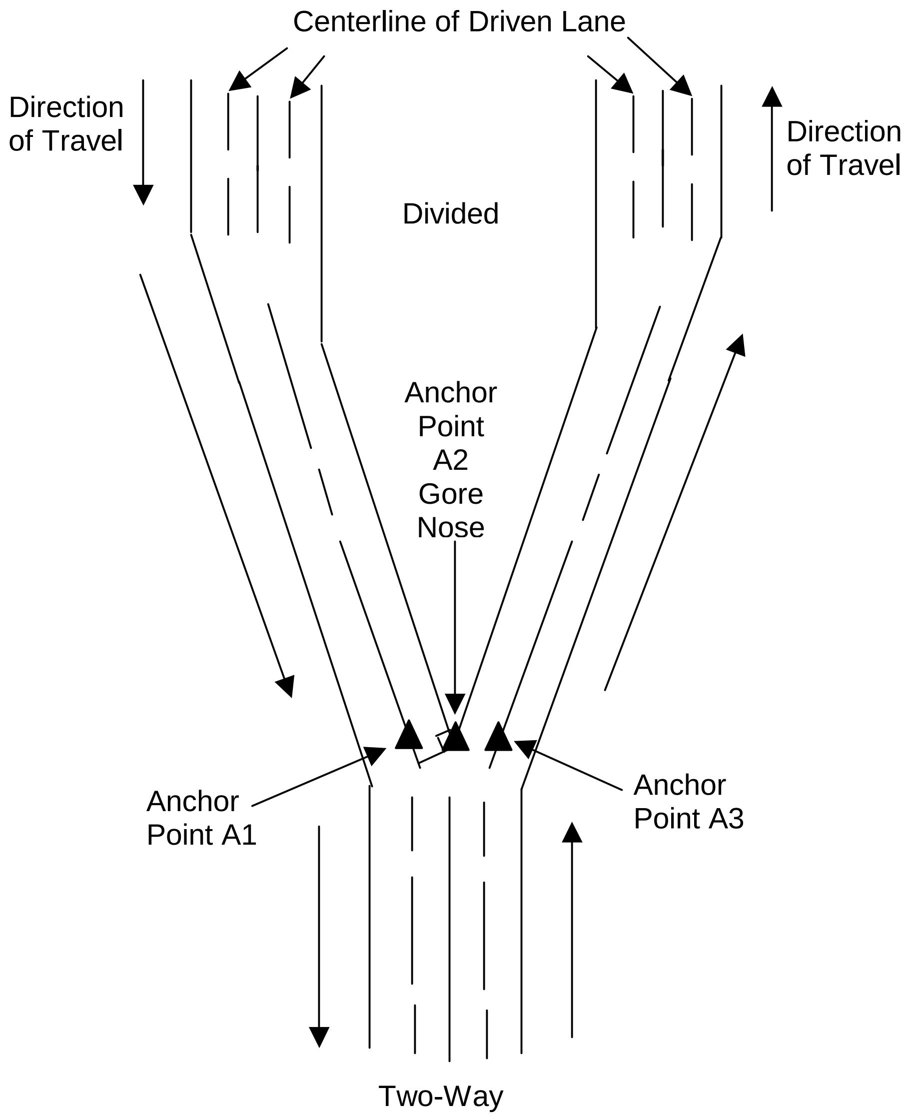
Business Rule: Where a two-way roadway becomes divided or a divided roadway becomes two-way, the anchor point is on one or both of the centerline(s) of the right-most lanes of the divided roadway at intersection(s) of perpendicular offset(s) from the gore nose (see Figure 11-6). The gore nose is defined as the intersection of the pavement edges. Some gore noses might be squared off or rounded.

Explanation: Determination of whether or not the anchor point appears on both of the divided roadway segments depends upon the outcome of first, second, and / or third order design for optimal configuration of the LRS. If the anchor point appears on both the divided roadway segments, there is still only one anchor point in linear space. The gore nose is a witness to this anchor point (Anchor Point 2 in the diagram) in the three-dimensional world and is coincident with it in linear space. Anchor Point 2 is also a witness point (RTK) for Anchor Point 1 and Anchor Point 3.

Business Rule: The HPMS definition of divided roadway will be adopted: “A divided facility is a roadway with 4 or more lanes and a median width of 4 feet or greater or a median type of positive barrier (median type code 2) or curbed (median type code 1).” (FHWA, 1998, pg I-1)

For the purposes of measurement, the gore nose should be identified and the anchor point location(s) should be estimated from the gore nose location. Distance measurements should begin / end at anchor points. Coordinates of the anchor point(s) should be determined, but will have little meaning, as a single linear anchor point can have two locations in two- or three-dimensional space. In some cases, the coordinates of the gore nose will have been determined by carrier phase kinematic GPS.

Figure 11-6 Anchor Points Where Roadways Become Divided

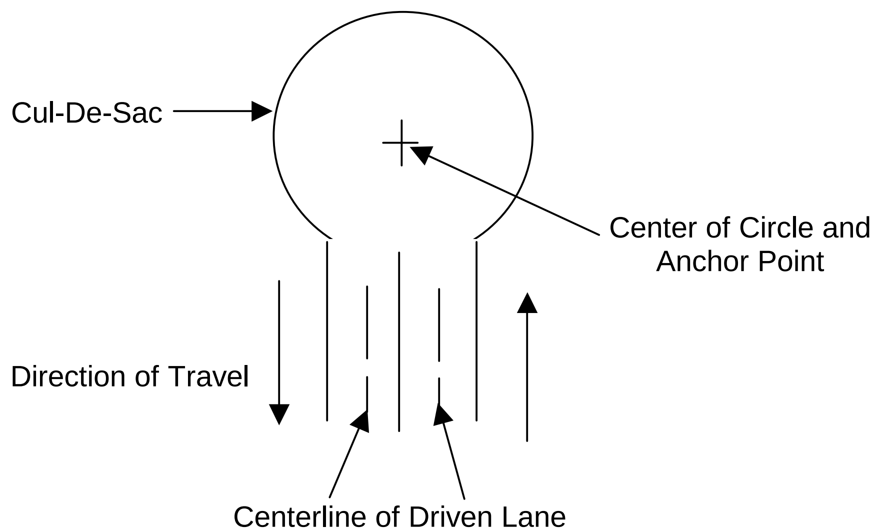


11.2.6. Cul-De-Sacs

Business Rule: The anchor point at a cul-de-sac is the center of the circle defined by pavement edges and or / curb and gutter. See Figure 11-7.

For the purposes of measurement, anchor point location is to be determined visually by estimating the center of the circle from pavement edges and / or curb and gutter. Distance measurements should begin / end at the anchor point. Coordinates of the anchor point should be determined.

Figure 11-7 Anchor Points at Cul-De-Sacs

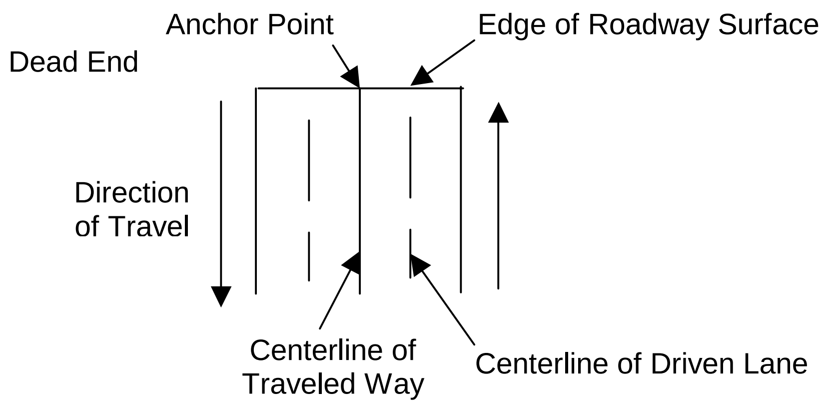


11.2.7. Dead Ends

Business Rule: The anchor point at a dead end is the intersection of the centerline of the traveled way with the edge of the roadway surface (all surface types). See Figure 11-8.

For the purposes of measurement, anchor point location is to be determined visually by estimating the centerline of the traveled way from its edges and finding the intersection of this centerline with the edge of the roadway at the dead end. Distance measurements should begin / end at the anchor point. Coordinates of the anchor point should be determined.

Figure 11-8 Anchor Points at Dead Ends



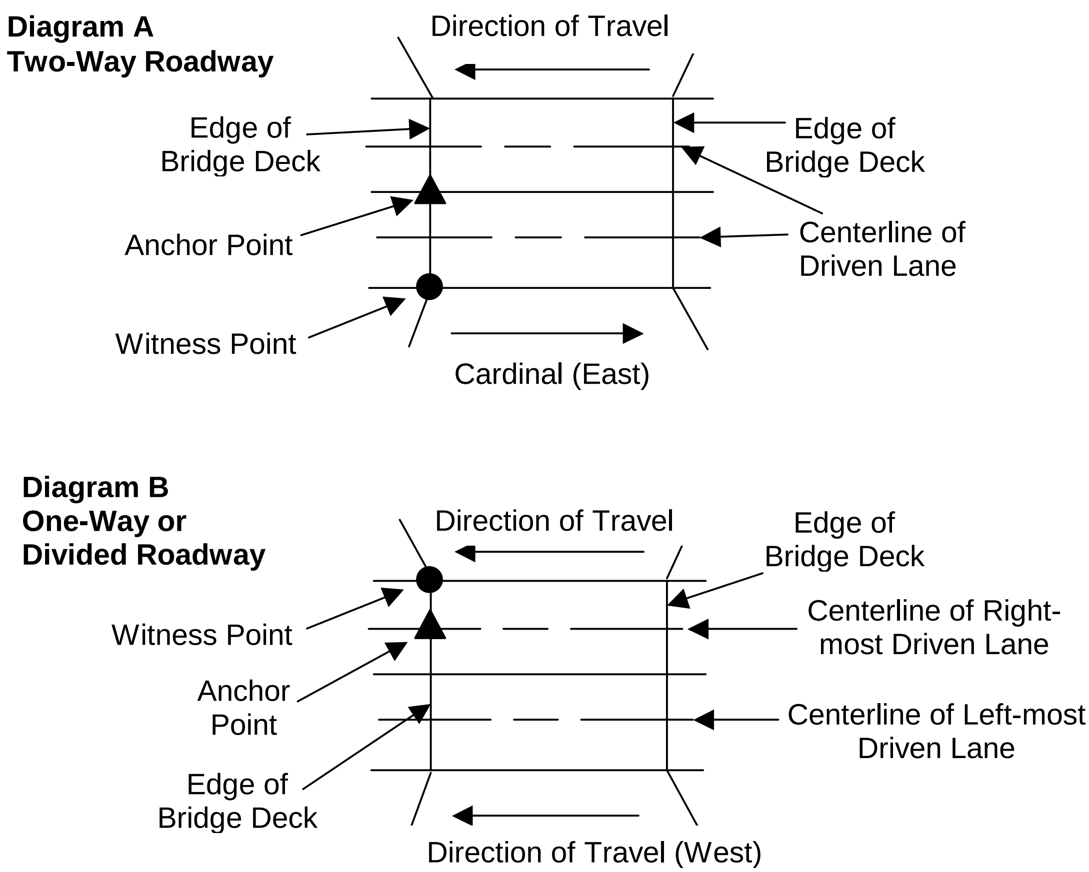
11.2.8. Bridges

Business Rule: For bi-directional roadways, an anchor point at a bridge is located at the intersection of the centerline of the roadway with the southernmost (first priority) or westernmost (second priority) edge of the bridge deck (see Figure 11-9, Diagram A).

Business Rule: For divided roadways or one-way roadways, an anchor point at a bridge is located at the intersection of the right-most driven lane with the southernmost (first priority) or westernmost (second priority) edge of the bridge deck (see Figure 11-9, Diagram B).

For the purposes of measurement, anchor point location is to be determined visually by estimating the centerline of the driven lane and then finding the intersection of this centerline with the edge of the bridge deck. Distance measurements should begin / end at the anchor point. Coordinates of the anchor point should be determined.

Figure 11-9 Anchor Points at Bridges



11.3. Statistical Analysis of Videolog Van DMI Measurements

Figure 11-10 is a histogram of differences between videolog DMI and low-resolution orthophoto anchor section distances. Also shown in Figure 11-10 is a histogram of a corresponding normally distributed random variable.

Figure 11-11 illustrates the cumulative distribution of both distance differences and a normal random variable with the same mean and standard deviation. The Kolmogorov-Smirnov test for distribution compares the maximum absolute difference in height between these two curves to a statistical criterion. If the maximum absolute height difference is greater than the criterion, the hypothesis that the distances differences are normally distributed is rejected at whatever level of significance was selected for the criterion. The value of the criterion is a function of the level of significance and the number of observed distance differences. For 161 distance differences and a 5% level of significance, the test criterion is 0.11. The maximum height difference between the two curves in Figure 11-11 is 0.09 and we fail to reject the hypothesis that the distance differences are normally distributed, thus validating the data for statistical analysis.

Figure 11-10 Histogram of Differences between Videolog DMI and Low-Resolution Orthophoto Distances

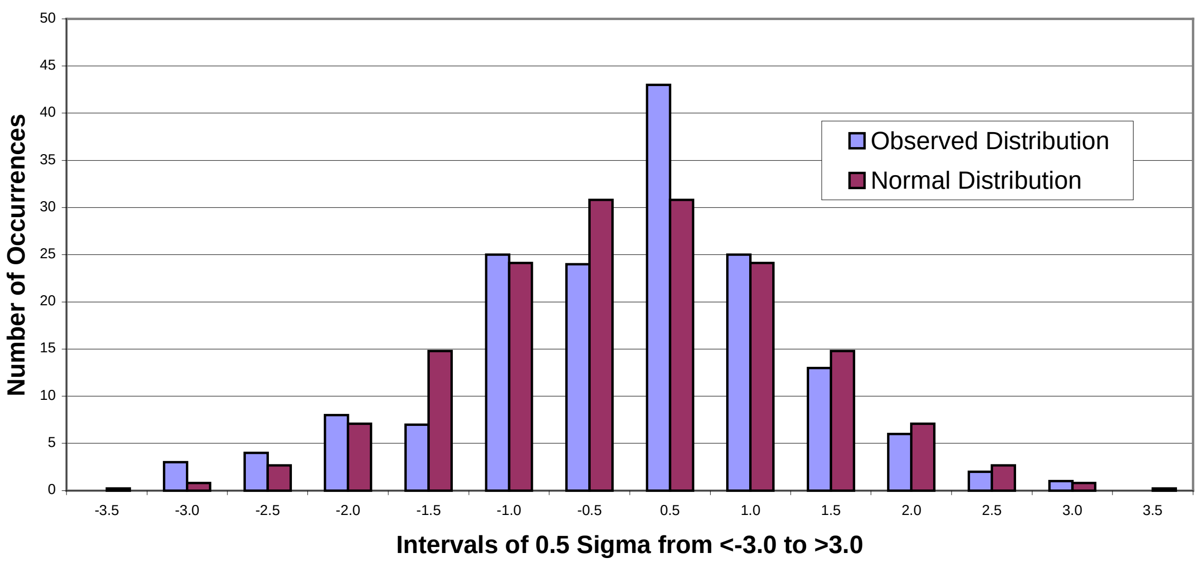
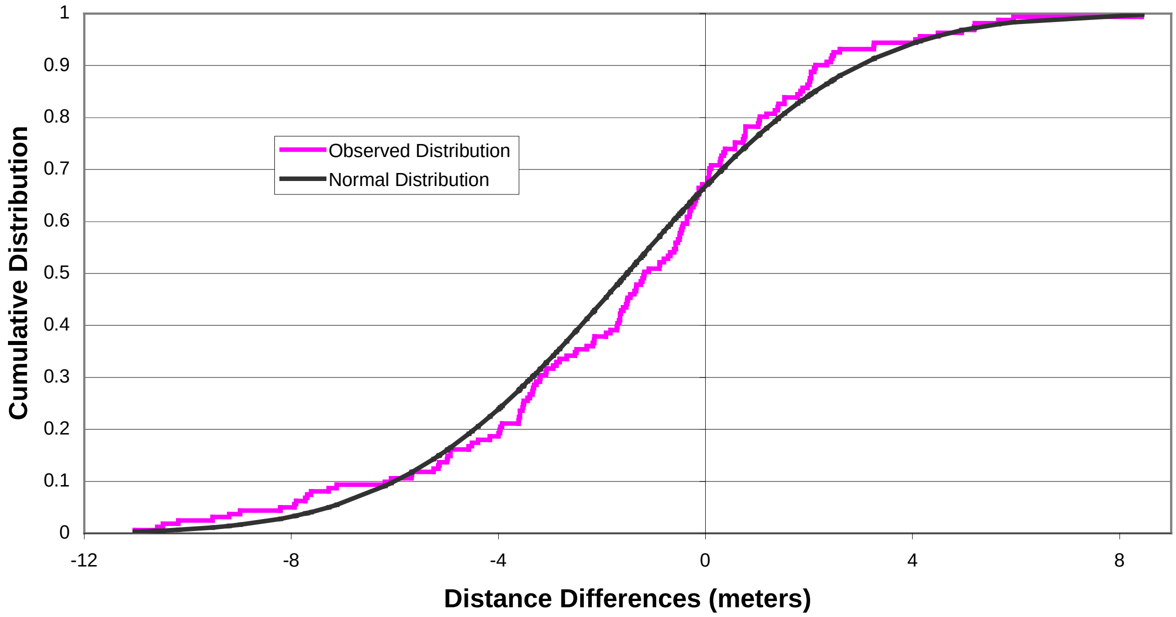


Figure 11-11 Cumulative Distribution of Differences between Videolog Van and Low-Resolution Orthophoto Distances



11.4. Statistical Analysis of Low-Resolution Orthophoto Measurements

The procedure used by the operators for digitizing anchor sections was to first establish the locations of anchor points and then snap to those locations each time the anchor point was revisited. Thus, there was a single measurement of each anchor point coordinate pair for each operator. Anchor point coordinates, as measured by operator 1, were compared to those measured by operator 2. Software was written that computed coordinate differences and their means and standard deviations for both the X and Y direction. Using a process similar to that for distance comparison, differences larger than three standard deviations from the mean were iteratively rejected until no such differences remained.

Initially, there were 427 anchor point coordinate differences. After 17 iterations, 145 (34%) had been rejected, leaving 282 coordinate comparisons. Positional differences as large as 100m were not uncommon among those that were rejected. Many of the rejected differences can be explained by previously stated reasons. Table 11-1 contains the mean and standard deviation of the differences in X, Y, and position for the final data set.

Table 11-1 Statistics for Low-Resolution Orthophoto Anchor Point Coordinate Differences (Operator 1 – Operator 2)

Statistic	Difference in X	Difference in Y	Difference in Position
Average	-0.2m	-0.1m	0.8m
Standard Deviation	0.6m	0.6m	0.5m

Figure 11-12 is a histogram of the number of occurrences of X coordinate differences at intervals of 0.5 times the standard deviation. A histogram of a normally distributed random variable with the same mean and standard deviation also appears in Figure 11-12.

Figure 11-12 Histogram of X Coordinate Differences between Operators 1 and Operator 2 Anchor Point Coordinates (Low-Resolution Orthophotos)

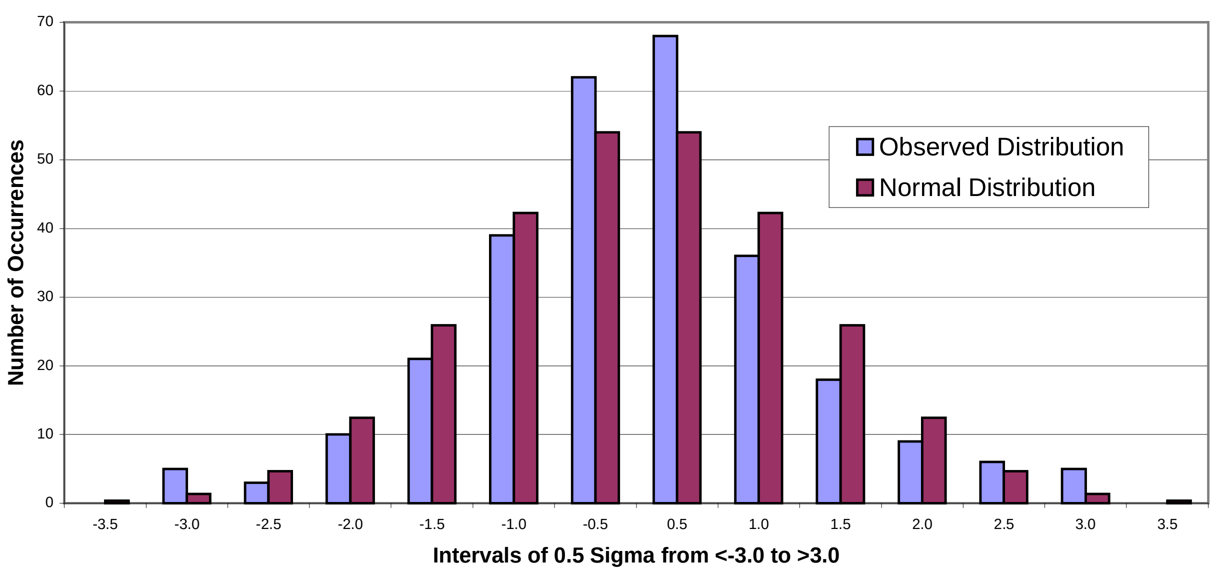


Figure 11-13 illustrates both the cumulative distribution of the X coordinate differences and the cumulative distribution of the associated normal variable. The Kolmogorov-Smirnov test criterion for 282 X coordinate differences at a 5% level of significance is 0.081. The maximum absolute difference in height between the two curves in Figure 11-13 is 0.067. Therefore, we fail to reject the hypothesis that the X coordinate differences are normally distributed.

Figure 11-13 Cumulative Distribution of X Coordinate Differences between Operator 1 and Operator 2 Anchor Point Coordinates (Low-Resolution Orthophoto)

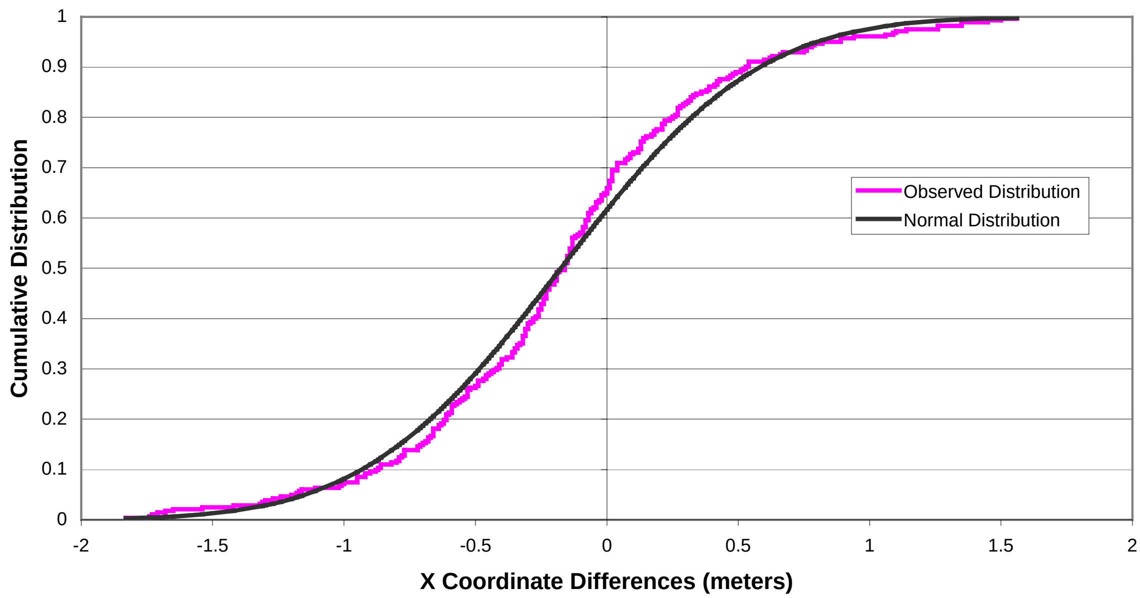


Figure 11-14 is a histogram of the number of occurrences of Y coordinate differences at intervals of 0.5 times the standard deviation. A histogram of a normally distributed random variable with the same mean and standard deviation also appears in Figure 11-14.

Figure 11-15 illustrates both the cumulative distribution of the Y coordinate differences and the cumulative distribution of the associated normal variable. The Kolmogorov-Smirnov test criterion for 282 Y coordinate differences at a 5% level of significance is 0.081. The maximum absolute difference in height between the two curves in Figure 11-15 is 0.036. Therefore, we fail to reject the hypothesis that the Y coordinate differences are normally distributed. This result, in conjunction with that for the X coordinate differences, validates the statistical analysis of the data.

Figure 11-14 Histogram of Y Coordinate Differences between Operator 1 and Operator 2 Anchor Point Coordinates (Low-Resolution Orthophotos)

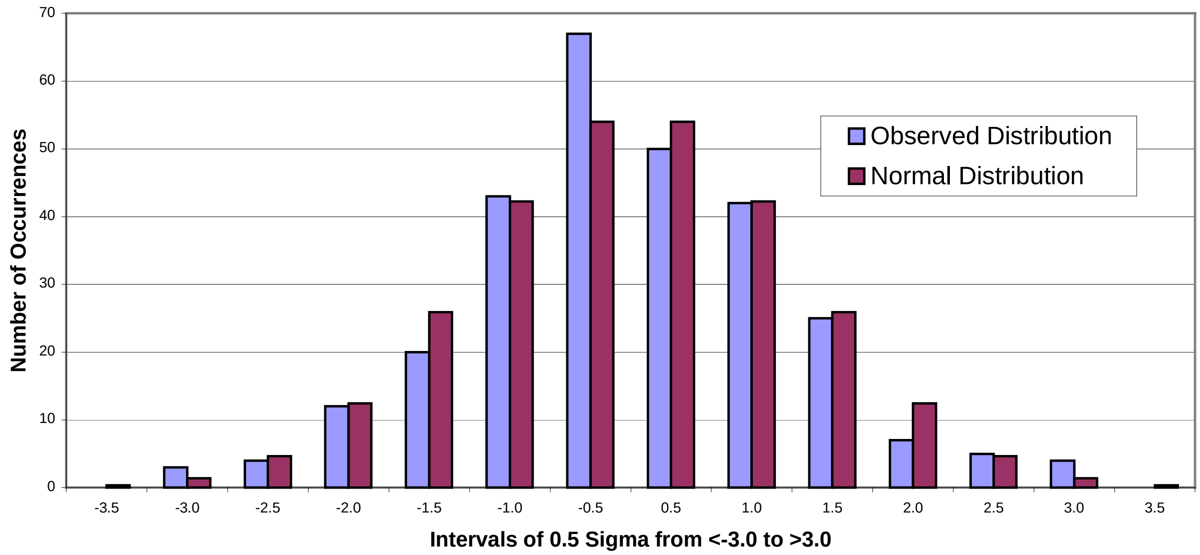
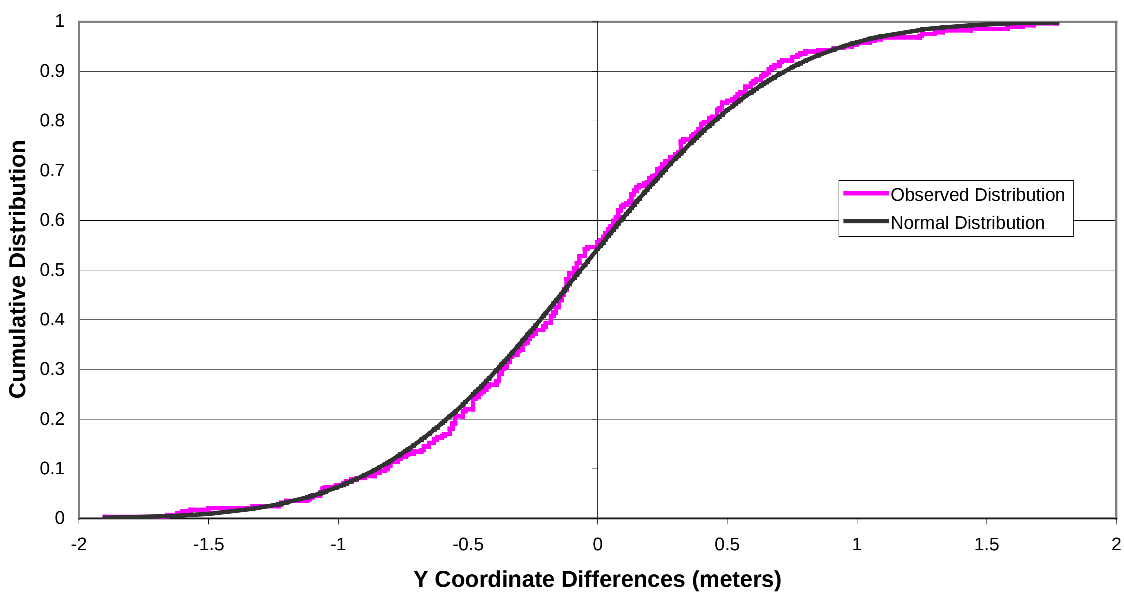


Figure 11-15 Cumulative Distribution of Y Coordinate Differences between Operator 1 and Operator 2 Anchor Point Coordinates (Low-Resolution Orthophotos)



In an effort to determine which of the operators' anchor point coordinates to use as a reference data set, both were compared to the kinematic GPS anchor point coordinates. It was not feasible to merely average the coordinates of the two operators to obtain a reference data set.

As demonstrated, the coordinate comparison had revealed some gross discrepancies between the two operators.

Table 11-2 contains the results of running the coordinate comparison software against the kinematic GPS coordinates and the coordinates of, first, operator 1, then, operator 2. The anchor point coordinate data set for operator 2 is not significantly different than that for operator 1 when compared to the kinematic GPS data set. However, it was known that operator 2 routinely consulted the field notes to help resolve ambiguities in identifying anchor point locations. For these reasons, the anchor point coordinates of operator 2 were selected as a reference data set for comparison with other methods.

Table 11-2 Results of Comparing Both Operators' Anchor Point Coordinate Data Sets to the Kinematic GPS Coordinate Data Set

Operator	One	Two
Number of Iterations	7	5
Number of Rejected Differences	12 (18%)	11 (16%)
Final Number of Differences	53	57
Average Difference in X	0.1m	0.1m
Standard Deviation in the X Differences	0.9m	0.7m
Average Difference in Y	0.0m	-0.3m
Standard Deviation in the Y Differences	0.7m	0.7m
Average Difference in Position	1.0m	0.9m
Standard Deviation in the Positional Differences	0.6m	0.5m

Figure 11-16 is a histogram of the number of occurrences of X coordinate differences (kinematic GPS-operator 2) at intervals of 0.5 times the standard deviation. A histogram of a normally distributed random variable with the same mean and standard deviation also appears in Figure 11-16.

Figure 11-17 illustrates both the cumulative distribution of the X coordinate differences and the cumulative distribution of the associated normal variable. The Kolmogorov-Smirnov test criterion for 57 X coordinate differences at a 5% level of significance is 0.18. The maximum absolute difference in height between the two curves in Figure 11-17 is 0.06. Therefore, we fail to reject the hypothesis that the X coordinate differences are normally distributed.

Figure 11-18 is a histogram of the number of occurrences of Y coordinate differences (kinematic GPS-operator 2) at intervals of 0.5 times the standard deviation. A histogram of a normally distributed random variable with the same mean and standard deviation also appears in Figure 11-18.

Figure 11-19 illustrates both the cumulative distribution of the Y coordinate differences and the cumulative distribution of the associated normal variable. The Kolmogorov-Smirnov test criterion for 57 Y coordinate differences at a 5% level of significance is 0.18. The maximum absolute difference in height between the two curves in Figure 11-19 is 0.10. Therefore, we fail to reject the hypothesis that the Y coordinate differences are normally distributed. This result, in conjunction with that for the X coordinate differences, validates the statistical analysis of the data.

Figure 11-16 Histogram of X Coordinate Differences between Low-Resolution Orthophoto (Operator 2) and Kinematic GPS Anchor Point Coordinates

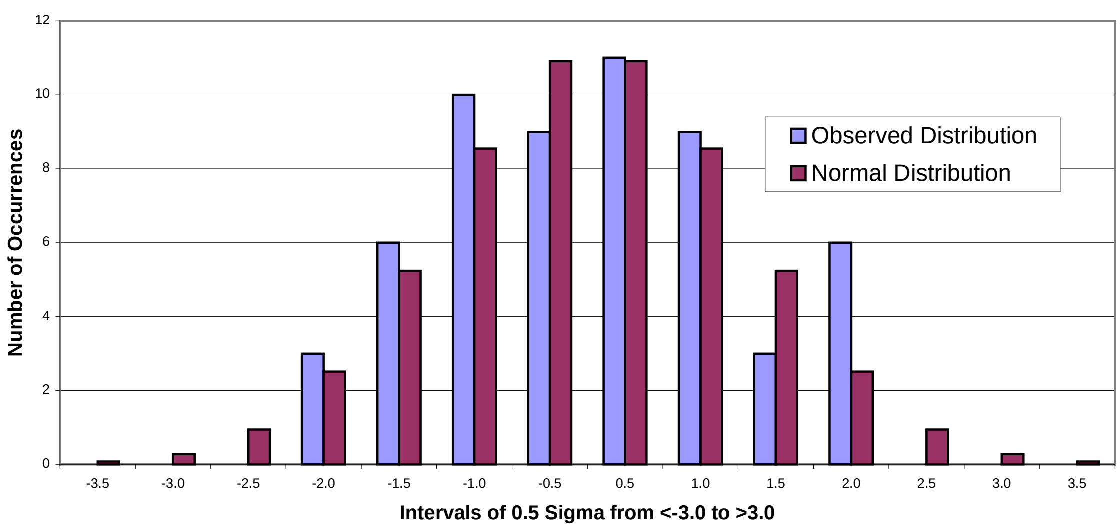


Figure 11-17 Cumulative Distribution of X Coordinate Differences between Low-Resolution Orthophoto (Operator 2) and Kinematic GPS Anchor Point Coordinates

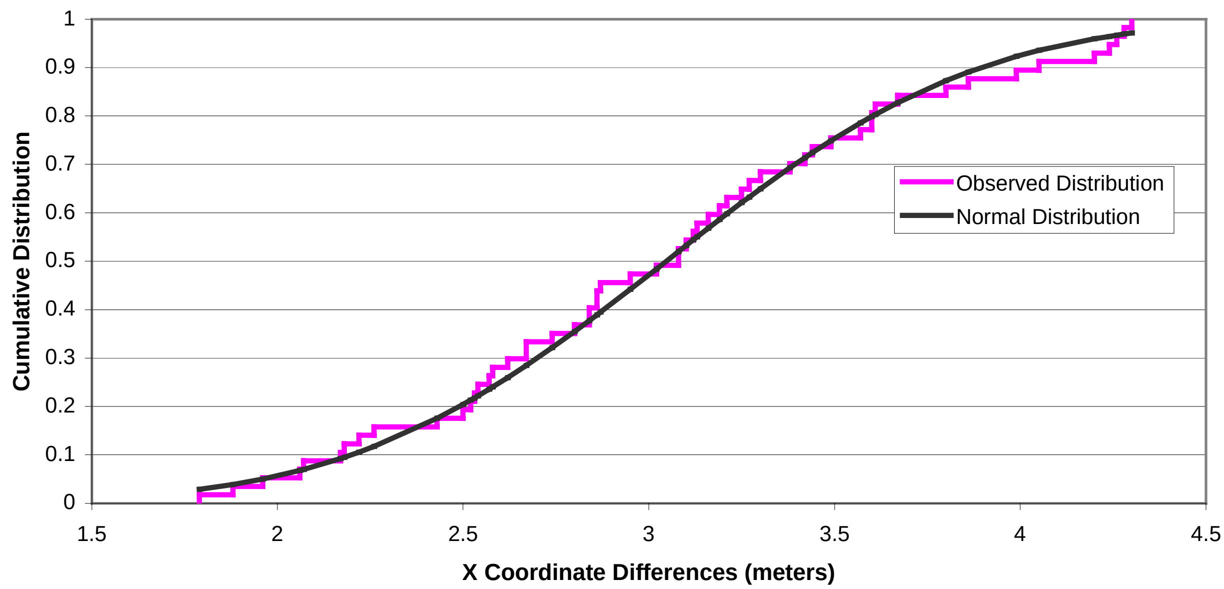


Figure 11-18 Histogram of Y Coordinate Differences between Low-Resolution Orthophoto (Operator 2) and Kinematic GPS Anchor Point Coordinates

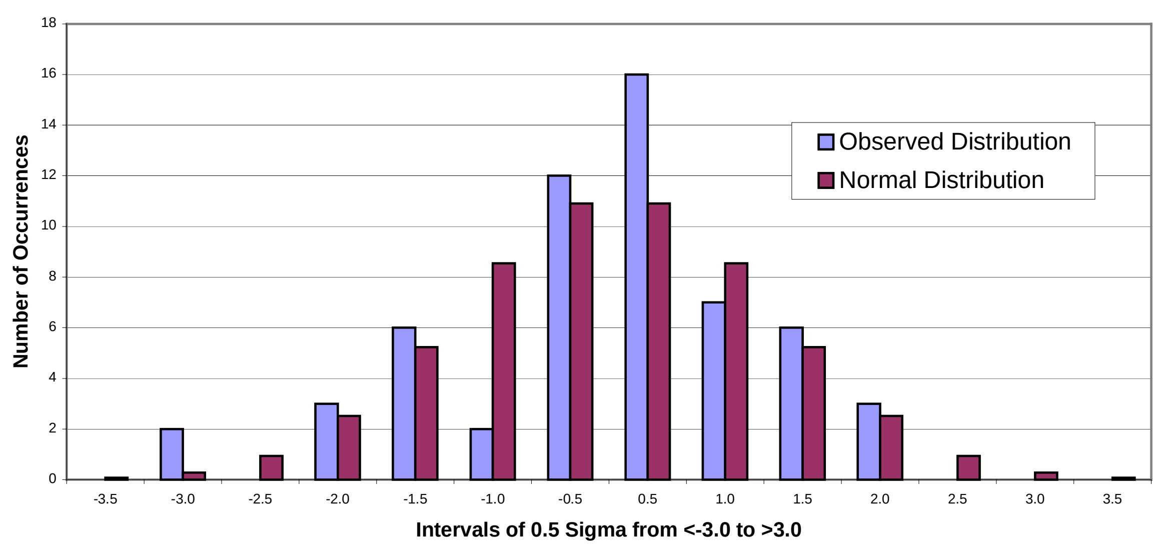
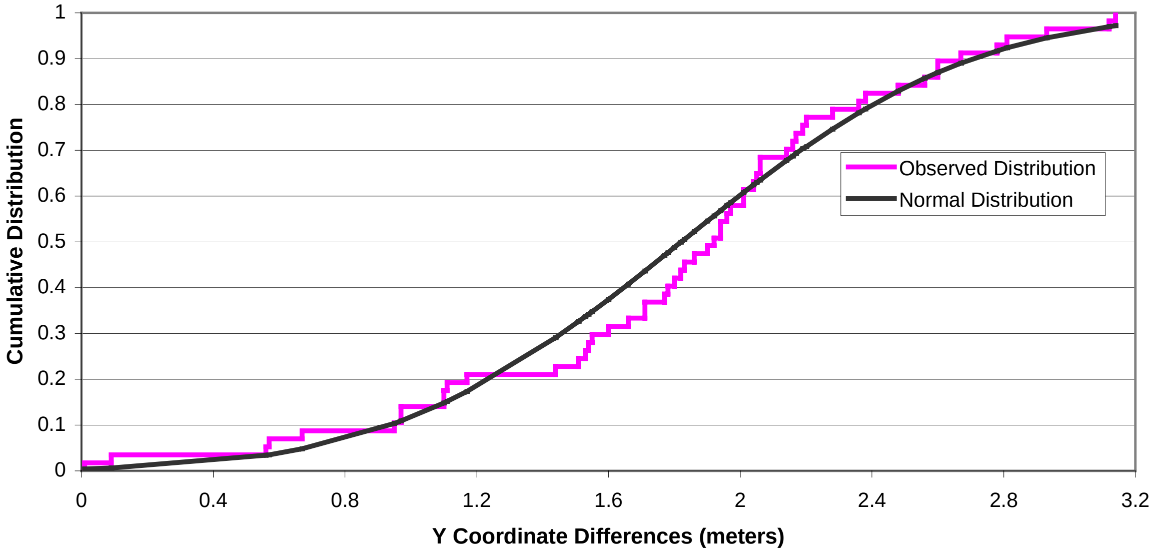


Figure 11-19 Cumulative Distribution of Y Coordinate Differences between Low-Resolution Orthophoto (Operator 2) and Kinematic GPS Anchor Point Coordinates



11.5. Estimation of Number of Datum Objects for Full-Scale Implementation

For the Field Pilot task area, Table 11-3 shows a breakdown, by DOT roadway system, of the number of anchor sections, number of roadway miles, and number of anchor sections per roadway mile. The number of miles for the primary system includes mileage for both directions for divided roadways. In the table, the number of anchor sections for the primary system does not include ramps (34). Since anchor sections are independent of the system, anchor sections that include GIMS segments from more than one system are not included in this table. Other anchor sections were also excluded from the table, as explained below.

Table 11-3 Miles per Anchor Section by Roadway System for the Pilot Area

System	# Anchor Sections	# Roadway Miles	# Miles per Anchor Section
Primary	16	62	4.0
Secondary	30	70	3.0
Municipal	82	32	0.5
Parks and Institutional	14	6	0.43

The primary system in Table 11-3 does not include Ramps. There were two anchor sections that contained both primary and municipal GIMS segments (IA 930 and IA 133). Because the Field Pilot area is only 4 miles in height, three anchor sections are substantially shorter than optimal design would have allowed. A length of 4 miles was used as a conservative estimate of the average length of a primary anchor section.

A number of Secondary anchor sections also contained Municipal GIMS segments. As discussed in the preceding section a number of secondary anchor sections were foreshortened because of the height of the pilot area. Based on these facts a conservative estimate of 3.0 miles was used for the average length of a secondary anchor section.

The Municipal System statistics only includes those anchor sections that were entirely within the city of Nevada. The city of Nevada has several features that caused some anchor sections to be shorter (two east/west rail lines and one major stream. Anchor sections in Ames were not used for the estimate. These anchor sections were selected to model unique terrain features and new subdivisions. Roughly half of mileage in Municipalities occur in cities larger than Nevada. It is believed that longer average anchor sections will be possible in these cities. Based on this and the fact that 20% of the municipal mileage for Nevada was on anchor sections that contained multiple GIMS systems, a conservative estimate of .5 miles was used for the average Municipal anchor section length.

The last column in Table 11-3 was used, along with mileage data for the full state of Iowa, to estimate the number of anchor sections, excluding ramps, which were counted separately. Although some pairs of ramps (e.g., at diamond intersections) might be spanned by anchor sections, each ramp was taken to include an individual anchor section for the purpose of this estimate.

Table 11-4 contains a breakdown, by roadway system, of the number of miles, number of miles per anchor section, and number of anchor sections for the State of Iowa. The primary system mileage in the table includes 2,090 miles estimated for both directions of divided highways. This number is equal to the total mileage of Iowa roadways with four or more lanes. It should be noted that some roadways with four or more lanes might not be divided.

Table 11-4 Estimated Number of Anchor Sections by Roadway System for the State of Iowa

System	# Roadway Miles	# Miles per Anchor Section	# of Anchor Sections
Primary	12,130	4.0	3,000
Secondary	89,180	3.0	30,000
Municipal	13,130	0.5	26,000
Parks and Institutional	460	0.43	1,000

The total number of anchor sections from Table 11-4 is 62, 000. This number includes the total number of ramps in Iowa estimated to be 2,000. The 12,000 primary system roadway miles, with 4.0 miles per anchor section, yields 1,500 8-mile anchor point spans. Given spans can be as long as 12 miles, this estimate is conservative.

The number of unique anchor points in the Field Pilot task area is 444. For 256 anchor sections, this yields 1.73 anchor points per anchor section. Applying this number to the estimated total number of anchor sections for Iowa yields 107,000 anchor points.

To provide conservative estimates 20% is added to each of the derived totals, yielding **75,000** anchor sections, **129,000** anchor points, and **1800** anchor point spans.

Estimates on datum maintenance also were created. Datum maintenance is driven by changes in roadway alignment. These alignment changes are driven by several factors, including but not limited to the construction of new roadways, expansion from two to four lanes, safety-driven intersection improvements, and errors or improvements in datum field measurement.

The Project Team could not locate data that describes only these types of changes on all roadway systems (primary, secondary, municipal, institutional). Iowa DOT anticipates that most new corridor development for the primary system is expected to be completed within the next four years. However, the Project Team assumes that secondary and municipal roadway changes will remain active, especially new development in growing urban areas and safety improvements in rural areas.

The Project Team decided to assume that approximately 1-2% of the roadway system has alignment changes each year. Given the datum statistics provided above, this means the following number of datum objects will require database maintenance each year: approximately **750-1500** anchor sections, **1,300-2,600** anchor points, and **18-36** anchor point spans.

11.6. Derivation of Elevation Factor for Field Pilot Area

Both ellipsoid and map projection distances, for the GPS coordinate strings coming from the videolog van, are automatically computed by Projection Manager / GeoMedia. Before these distances can be compared to those coming from the DMI in the van, they must be converted to ground distances. This is most easily done by multiplying the ellipsoid distances by a single elevation factor for the entire Pilot area. Use of a single elevation factor is appropriate because the Pilot area has very little change in elevation across its extent.

The elevation factor is a ratio of the ellipsoid radius plus the mean ellipsoid height of the Pilot area to the ellipsoid radius. That is,

$$F_E = (Radius + MeanHeight) / Radius \tag{8}$$

where

$F_E =$ the elevation factor,

$MeanHeight =$ the mean ellipsoid height of the pilot area, or

$MeanHeight =$ 278m (based on Iowa DOT's statement of 308m for mean project elevation and 30m mean geoid undulation at HARN stations),

$Radius =$ Mean ellipsoid radius at the latitude of the pilot area, or

$Radius = \sqrt{\rho \upsilon}$,

$\rho =$ Radius of curvature in the meridian = $a(1-e^2)/(1-e^2\sin^2\phi)^{\frac{3}{2}}$,

$\upsilon =$ Radius of curvature in the prime vertical = $a/(1-e^2\sin^2\phi)^{\frac{1}{2}}$,

$e =$ Eccentricity of the ellipsoid = $\sqrt{(a^2-b^2)/a^2}$,

$\phi =$ Mean latitude of the pilot area = 42 degrees,

$a =$ Semi-major axis of the ellipsoid = 6,378,137m (WGS84),

$b =$ Semi-minor axis of the ellipsoid = 6,356,752m (WGS84).

Then

$$e^2 = (6378137^2 - 6356752^2)/6378137^2 = 0.006694478,$$

$$\nu = 6378137/(1 - 0.006694478\sin^2(42^o))^{\frac{1}{2}} = 6387717 \text{ m},$$

$$\rho = 6378137(1 - 0.006694478)/(1 - 0.006694478\sin^2(42^o))^{\frac{3}{2}} = 6364030 \text{ m},$$

$$Radius = \sqrt{(6364030)(6387717)} = 6375863 \text{ m, and}$$

$$F_E = (6375863 + 278)/6375863 = 1.000043536 \tag{9}$$

Each of the ellipsoid distances should be multiplied by 1.000043536 to obtain equivalent ground distances.

11.7. Least Squares Adjustment

Software was developed for performing a least squares adjustment of these data. The software solves the system of normal equations:

$$(A^T PA)X = A^T PL \tag{10}$$

where A is a matrix of coefficients that relate the measurements (L) to the unknowns (X) and P is a weight matrix (the inverse of the variance-covariance matrix) for the observations. A^TPA is positive definite and extremely sparse. The software includes an equation optimizer that should allow adjustment of very large systems during future implementations of the linear datum. The software can be used to adjust datum distances measured by any method. The software also computes the variance-covariance matrix of the unknowns, given by

$$\Sigma = \sigma_0^2 (A^T PA)^{-1} \tag{11}$$

where σ_0^2 is referred to as "the reference variance" and is given by

$$\sigma_0^2 = V^T PV /(m-n)\,. \tag{12}$$

Here, m is the number of measurements, n is the number of unknowns, m-n is the degrees of freedom, and V is a vector of residuals in the measurements, given by

$$V = AX - L\,. \tag{13}$$

The square root of the reference variance is often referred to as the "standard deviation of unit weight". In order for a least squares adjustment to be acceptable, the value of σ_0^2 must be close to 1. Otherwise, there are blunders in the measurements, P is estimated incorrectly, or both. Any least squares adjustment typically involves a number of attempts, with blunders being identified and removed and with modifications being made to P, until an acceptable value of σ_0^2 is produced.

The weight matrix, P, includes estimates for the standard deviations in the measurements. For the Videolog Van DMI, final model used for computing the standard deviation of a measurement for the purpose of weighting was

$$\sigma = \sqrt{2 * 2.3^2 + ((L/5280) * 0.3048)^2} \tag{14}$$

where L is the measured distance in feet and σ is in meters. The uncertainty is ± 1 ft per mile in a DMI measurement is taken from published information on the capabilities of the equipment. The model includes a pointing error of ± 2.3m at each end of a measurement. Pointing error is associated with the inability to exactly position the vehicle over or along side the anchor point at each end of a measurement.

The modeled standard deviation in a given measurement for least squares adjustment of the videolog van GPS/INS data was identical to that used for the videolog van DMI data.

The modeled standard deviation in a given measurement for least squares adjustment of the low-resolution orthophoto data was

$$\sigma = \sqrt{2 * 2.1^2 + ((L/5280) * 0.3048)^2} \quad (15)$$

similar to that of the DMI distances, where L is the measured distance in feet and σ is in meters. The pointing uncertainty of ± 2.1m was derived from a value determined experimentally. Each operator was asked to repeat digitize the same set of ten anchor points, selected for their variability in visual character, 15 times per point. The RMS positional discrepancy among these points was 2.0m. During the least squares adjustment of the digitized distances, this experimental value had to be relaxed only 0.1m to achieve an acceptable solution.

11.8. References

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