MONTANA DEPARTMENT OF TRANSPORTATION SURVEY MANUAL

Montana Department of Transportation

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MONTANA DEPARTMENT OF TRANSPORTATION

SURVEY MANUAL

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Chapter 1
General

1.1 INTRODUCTION

This manual was developed to provide statewide uniformity in surveying practices, to establish and maintain survey standards, and to improve the overall efficiency of the Department's survey function. Both Department of Transportation personnel and consultants retained by the Department should adhere to the standards, procedures, and specifications given in this manual. The Engineering Project Manager (EPM), the consultant, or the land surveyor should ensure their surveys conform to this manual. For the purpose of this manual, the term "survey crews" refers to both MDT employees and consultants and their agents.

If the surveying requirements of this manual conflict with the requirements of contract documents, then the contract prevails.

1.1.1 DESCRIPTION OF SURVEYING

Surveying is performed to determine the relative positions of points on or near the earth's surface. It involves making measurements and calculations to provide essential information needed for planning highway routes, acquiring land, designing and constructing highways and other related activities.

1.1.2 CHAPTER SUMMARIES

The Surveying Manual is organized in to twelve chapters and five appendices.

Chapter 1. An introduction to the Survey Manual. Includes information regarding public relations, right of entry, safety, and effective date.

Chapter 2. Survey Datums and Coordinate Systems. Addresses the datums and systems used as references for surveys. It includes discussions of vertical and horizontal datums, perpetuation of National Geodetic Survey (NGS) bench marks, local coordinate systems, the Montana state plane coordinate system and other related topics.

Chapter 3. Surveying Equipment, Measurements and Errors. Deals with common survey equipment, the care and maintenance of surveying equipment, general observation techniques, the use of NGS calibrated base lines, horizontal and vertical measurements, and errors.

Chapter 4. Errors and Maximum Closures. Discusses accuracy and precision, types and sources of errors, maximum closures associated with differential levels, conventional control surveys, and celestial observations.

Chapter 5. Conventional Control Surveys. Describes reports that are developed prior to preliminary surveys, conventional control traverse(s) traverse configurations, numbering (designation) of control marks, abstracts, differential levels, basis of bearing, angular and distance measurements, traverse adjustments, and deliverables.

Chapter 6. Secondary Traverses, Radial Surveys and Cadastral Surveys. Describes secondary traverses their use and adjustments, radial survey methods and their use, purpose and requirements of a cadastral surveys, the involvement of the land surveyor, and deliverables.

Chapter 7. Reserved for Aerial Photography and Photogrammetric Surveys.

Chapter 8, Global Positioning System. Discusses control surveys, project control requirements, vertical control, network configurations and adjustment, cadastral surveys and specifications, and deliverables.

Chapter 9. Reserved for Data Collectors.

Chapter 10. Construction Surveys. Covers survey methods, horizontal control and vertical control, staking, perpetuation of NGS bench marks, right of way monumentation, clearing and grubbing, earthwork, bridge surveys, drainage facilities, and curb and gutter staking.

Chapter 11. Hydraulic Surveys. Describes survey data required, HYD-1 in general and specific sections of the HYD-1 form, urban storm drainage, and culvert surveys.

Chapter 12. Field Notes. Outlines the importance of field notes, information that should be included in preliminary and construction field notes, and disposition of field notes.

The five appendixes cover the subjects as indicated by their titles:

Appendix A. Sample Notes

Appendix B. References

Appendix C. Glossary. Definitions, abbreviations and conversions.

Appendix D. Basic Trigonometry. Solutions of right and oblique triangles.

Appendix E. Curves. Circular, spiral, and vertical curves.

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1.2 MAINTENANCE AND UPDATING OF MANUAL

Everyone who has access to this manual should review the content and format, and submit recommendations for improvements. The primary intent is to provide all survey crews with standard procedures. All persons should feel free to make suggestions. These suggestions will improve the next edition. This manual will be updated periodically to keep it current. The suggestions should be submitted on the MDT Survey Manual Revision Request Form 1-1 and can be accompanied by drawings or notes in any form.

1.2.1 MDT SURVEY MANUAL REVISION PROCESS

Revisions to the *MDT Survey Manual* will be submitted and reviewed according to the following process:

- Proposed revisions are to be submitted to the Supervisor of Photogrammetry & Survey using the Survey Manual Revision Request Form.
- A six-person review committee, selected by the Supervisor of Photogrammetry & Survey will meet as necessary to review the proposed changes and make revisions.
- The review committee will meet with the Supervisor of Photogrammetry & Survey and the Highways Engineer to submit their recommendations and to determine if the proposed revisions will be incorporated into the manual.
- If the manual will be revised as recommended, the Supervisor of Photogrammetry & Survey will distribute a memo describing the revision.

1.2.1.1 Review Committee

The review committee will consist of six members selected by the Supervisor of Photogrammetry & Survey. The committee will include the Land Survey Coordinator and one member from each of the following areas: Hydraulics, District Construction, District Survey, Data Collection, Photogrammetry. With the exception of the Land Survey Coordinator, one or more members will be replaced each year and members will serve no more than three consecutive years. The Land Survey Coordinator will be a permanent member of the committee and will serve as the committee chairperson.

The review committee will be responsible for the following:

- reviewing and recommending changes to the *MDT Survey Manual*
- providing all updates for the *MDT Survey Manual*

- distributing the manual
- maintaining a library of all revisions to the manual in chronological order.

1.3 ASSISTANCE

For assistance or answers to questions concerning interpretation of the manual, contact the Photogrammetry and Survey Section prior to the survey. The Photogrammetry and Survey Section or the District land surveyors can be contacted for assistance concerning general survey questions.

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1.4 PUBLIC RELATIONS

1.4.1 IMPORTANCE

The Montana Department of Transportation (MDT) is a public service organization, and each of its employees and consultants are representatives of the Department. Relationships with the general public, property owners, media, utility and railroad companies, governmental agencies, contractors and fellow workers can have a beneficial or adverse effect upon a project.

Survey crews are one of the most visible groups representing the Department, and their conduct on the job and contacts with the general public, property owners and media representatives in particular are of extreme importance in the maintenance of good public relations. Cultivate good relationships and do your best to avoid antagonistic encounters.

1.4.2 RIGHT OF ENTRY

Prior to entry on private land, written permission will be obtained by the Department or its agents from either the owner of record and/or the lessee of the property. The original signed form will be retained in the District's preliminary engineering file. The signed form will be scanned into a PDF document and placed on the Document Management System (DMS). Consultant's will provide Consultant Design the original signed form. Prior to actual entry on the property, it may be advisable to contact the landowner.

1.4.3 PROPERTY OWNERS

During the course of actual survey activities, the survey crews have day-to-day contact with the landowner. The survey crews should strive to maintain the best possible working relationships with property owners. Prior to any surveying activity, every crew member involved should be thoroughly briefed on the information that can be released at that time and who should be contacted for a more detailed explanation.

Most of the questions that a survey crew is asked can be anticipated and discussed beforehand with the entire crew. The answers should always be as concise as possible without speculation or guesswork. The survey crew or their supervisor should insure that an adequate answer is given to a landowner's question as soon as practical.

1.4.4 MEDIA

It is not the responsibility of the survey crews to discuss Department business with the media. If approached by media representatives, refer them to the district office. Throughout your contacts with media representatives, conduct yourself so as to maintain a good impression of the Department.

1.5 UNITS OF MEASUREMENT

Reference should be made to the Preliminary Field Review (PFR) to determine the coordinate system associated with the project. Project coordinates will be either English (feet) or metric units. All survey measurements (horizontal and/or vertical) will be made in the units specified in the Preliminary Field Review. When conversions of coordinates are necessary, the conversions shall be made in accordance with statutes.

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1.6 SAFETY

The Department has established safety standards in the work place. These standards are for the protection of MDT employees, their agents and the traveling public. Safety policies and procedures, including the MDT *Employee Safety Policy & Procedure Manual* and the *MT Operations Manual*, are available from the Organizational Development Bureau. The *MT Operations Manual* is presently available online.

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1.7 SIGNING FOR SURVEY CREWS

The Montana Department of Transportation requires signing when conducting surveys on or near roadways. For signing guidelines, refer to the appropriate sections of the *Manual on Uniform Traffic Control Devices,* current MDT detailed drawings and other appropriate Department policies.

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1.8 REFERENCES

This manual is not intended to be used as a substitute for surveying textbooks, other policies or standards from other disciplines, including but not limited to, Bridge, Road Design, Hydraulics, Environmental and Safety. See Appendix B for a list of textbooks and other references.

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1.9 EFFECTIVE DATE

The requirements contained in this manual will become effective upon Department approval and publication. The most current version of the Surveying Manual should be obtained prior to beginning any survey activities.

Projects having certain phases of preliminary surveys completed will adhere to the Surveying Manual in effect at the time of that survey. All new surveying activities will adhere to the requirements contained in the most current version of the Surveying Manual. For example, if bench levels are completed but the control traverse is not, the control traverse will be surveyed according the specifications in the most current manual.

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Survey Manual Revision Request Form

Identification

Date Submitted: ____________________

Section To Be Revised: ____________________

Section Title: ____________________

Page Number(s): ____________________

Description of Revision

Proposed revision:

____

____

____

List other sections of the manual that would be affected by the revision:

____

____

____

Reason For The Revision

____

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Form 1-1
Survey Manual Revision Request Form

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Chapter 2

Survey Datums and Coordinate Systems

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Chapter 2

Survey Datums and Coordinate Systems

A datum is a quantity or set of quantities that serve as a reference or base for other quantities. Historically the Montana Department of Transportation has used a variety of different datums such as assumed, record [as built, General Land Office (GLO), etc], or geodetic. The preferred horizontal datum for new Department projects is the North American Datum of 1983 (NAD83). Refer to Chapter 8 for additional information. The preferred vertical datum for new Department projects is the North American Vertical Datum of 1988 (NAVD88).

2.1 VERTICAL DATUMS

Elevations for engineering projects must be referenced to a single vertical datum. This datum can be based on a standard such as the National Geodetic Vertical Datum of 1929 (NGVD29), NAVD88, or the vertical datum can be assumed. The use of an assumed vertical datum should be restricted to unique situations, and must be approved by the Project Manager as defined in the OPX2 Project Scheduler. The Project Manager may consult with the Photogrammetry & Survey Section.

2.1.1 NGVD 1929

The majority of the work performed by the Department was referenced to mean sea level datum. This datum was established by the United States Coast and Geodetic Survey (USC&GS) and is referred to as NGVD29. The published elevations of their bench marks established before the mid-1980s were based on this datum. Elevations are now provided by the National Geodetic Survey (NGS), which is the successor to the USC&GS. The United States Geological Survey (USGS) also publishes elevations of their bench marks. USGS bench marks are relative to NGVD29. This vertical datum may continue to be used by the Department, but should only be utilized in unique situations. The use of this datum must be approved by the Project Manager. The Project Manager may consult with the Photogrammetry & Survey Section.

2.1.2 NAVD 1988

NAVD88 was established by NGS in the early 1990's. This datum adjustment held fixed the height of the primary tidal bench mark, referenced to the new International Great Lakes Datum of 1985 local mean sea level height value, at Father Point/Rimouske, Quebec, Canada. All new projects should utilize NAVD88. There may be exceptions to this such as isolated projects that do not involve a designated floodplain. NGS datasheets may be obtained on line at www.ngs.noaa.gov. NGS provides various methods to search their database such as a radial search, a rectangular search, station name, USGS quad map, county, etc. Figure 2-1 is a sample datasheet that was obtained from the NGS website. Datasheets list elevations and heights. The height values (geoid and dynamic) should not be used. Differential levels in conjunction with these bench marks should use either the metric or the foot elevation (NAVD88).

2.1.3 CONVERSION BETWEEN NGVD29 AND NAVD88

It may be necessary to convert a published NGVD29 elevation to a NAVD88 elevation. The most common method is to use NGS's program, Vertical Conversion (VERTCON). This program may be downloaded or run interactively from their website. VERTCON computes the modeled difference in orthometric height between NAVD88 and NGVD29 for a given location specified by latitude and longitude. The estimated accuracy (one sigma) of this conversion is 0.07 feet (0.02 meters).

Projects that encompass a designated floodplain must include the vertical shift from NGVD29 to NAVD88. The Hydraulics Section shall be provided this information. In Montana, NAVD88 elevations are higher than NGVD29 elevations. Refer to Figure 2-1. The NAVD88 elevation is 2429.39 feet (740.479 meters). The superseded survey control information indicates the NGVD29 elevation is 2427.51 feet (739.907 meters). The vertical shift from NGVD29 to NAVD88 is therefore +1.88 feet (0.572 meters).

Refer to Chapter 8 for additional information associated with the conversion between vertical datums.

2.1.4 OTHER VERTICAL DATUMS

Some urban projects may be referenced to a local, city or county datum. If a local datum is used, the field notes and plans must clearly state the origin of the datum.

An assumed datum may be used on small, remote projects where the expense to connect to a known datum is not justified. The Engineering Project Manager (EPM) or the consultant should consultant the Project Manager to verify that an assumed datum is acceptable. The assumed elevations should be large enough to avoid negative elevations but small enough so that they will not be confused with NAVD88 (eg 500.00 feet or 500.000 meters).

2.1.5 MONUMENTATION

Monumentation for both USC&GS and NGS bench marks have varied over the years, and many of the original bench marks have been disturbed or destroyed. Most of the bench marks are bronze disks. The disks may be set in standard concrete monument posts or embedded in concrete headwalls, bridges or rock outcrops. The latest style of bench mark being installed by NGS is a stainless steel rod driven to substantial refusal and protected by a covered aluminum box. The lid of the box must be lifted to access the datum point (point of reference). An example of the new NGS vertical control mark is shown in Figure 2-2.

2.1.6 PERPETUATION OF BENCH MARKS

The majority of level lines in the country run along highways or railroads because they provide easy access to the lines. Highway departments are major users of these level lines and major destroyers of bench marks through their construction activities.

The perpetuation of bench marks is in the best interest of the public and private engineering and land surveying firms. The Department should make efforts to protect and perpetuate all existing, published bench marks. It is the responsibility of the EPM to preserve existing USC&GS, USGS and NGS bench marks. Two situations exist:

- The original mark is present but will be destroyed and there is time to set a new replacement mark prior to the destruction of the original mark.
- The original mark will be destroyed prior to the establishment of a new mark, or the new mark will be set in a new structure such as a bridge.

The following is a brief description of the general requirements to establish a RESET mark. The survey crew should refer to the detailed instructions “Bench Mark Reset Procedures” which is located at http://www.mdt.mt.gov/publications/manuals.shtml#survey. These instructions are specific to vertical marks that are included in NGS’s database. In addition, these instructions also provide guidelines to replace a destroyed bench mark along an existing level line while maintaining the original vertical order accuracy. The reset procedures given in the instructions, and the general guideline below will result in the

RESET mark having a third order vertical accuracy regardless of the original mark vertical accuracy.

2.1.6.1 Reset of a New Mark is Possible Prior to Destruction of Original Mark

In this case, a new monument (mark) is set per NGS specifications. This monument will be either a stainless steel rod with an aluminum cover (refer to Figure 2-2), a disk epoxied into a drilled rock outcrop, a disk epoxied into a substantial structure such as a bridge, or a disk embedded into a new concrete post. If the disk is embedded into a new concrete post, the excavated hole should be approximately 4 feet deep and 1 foot in diameter (belled out at the bottom) and then filled with concrete. The new aluminum cover or the new disk will be stamped with the original mark designation plus the word RESET and the year set. For example, assume NGS bench mark V 62 will be destroyed. The newly set disk would then be stamped V 62 RESET and the year. The new mark should be set in a secure location and as near to the original monument as possible. In most cases, the new mark can probably be set within 460 feet (140 meters) of the original mark. Once the new mark has been set, the field crew should complete Form 2-1 (Report on Relocation of a Reset Bench Mark) and Form 2-2 (Station Description). Differential levels are run from the original mark to the new mark and from the new mark back to the original mark. The reset mark must be turned through. An intermediate foresight to the reset mark is not acceptable. These observations should be recorded on Form 2-3. Refer to Appendix A for an example of the completed forms. Photogrammetry & Survey has access to stainless steel rods, aluminum covers, and NGS vertical control disks. Contact the Photogrammetry & Survey Section for necessary supplies and assistance.

The completed information on Form 2-1 through Form 2-3 and any original field notes associated with the resetting of the bench mark should be sent to the Photogrammetry & Survey Section for submittal to the State Geodetic Advisor. In addition, if a digital level was utilized, the raw file should also be included on disk and submitted with the forms and original field notes.

2.1.6.2 Reset of New Mark is Not Possible Prior to Destruction of Original Mark

In this case, a minimum number of three vertically stable reference points or Temporary Bench Marks (TBM's) are required. The TBM's must maintain their elevation until their elevations can be transferred to the new mark. The three TBM's must be established outside the estimated construction limits and they must be fully described in field notes. A closed differential level loop is then established between the original mark and each TBM. Form 2-3 is completed for each TBM.

Once construction has been substantially completed in the vicinity of the TBM's, the new reset mark (stainless steel rod or NGS vertical disk) can be set. Differential levels are then run between each previously established TBM and the reset mark. Form 2-3 is completed for each closed differential level loop between each TBM and the reset mark.

The completed information on Form 2-1 through Form 2-3 and any original field notes associated with the resetting of the bench mark should be sent to the Photogrammetry & Survey Section for submittal to the State Geodetic Advisor. In addition, if a digital level was utilized, the raw file (from original mark to the TBM's and from the TBM's to the new mark) should also be included on disk and submitted with the Forms and original field notes.

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2.2 HORIZONTAL DATUMS

The vast majority of Department projects will have a control survey. The control survey will be established by conventional methods (Chapter 5) or by the Global Positioning System (Chapter 8). The primary purpose of a control survey is to establish coordinates of a series of semi-permanent monuments throughout the limits of the project. All additional coordinates obtained by survey such as engineering surveys, hydraulic surveys, cadastral surveys, staking of the designed project, staking of right-of-way, etc are then relative to the coordinates established during the control survey. The coordinates associated with the control survey may be relative to a local (assumed) coordinate system, or relative to state plane coordinates. The Preliminary Field Report (PFR) will generally indicate the coordinate system to be used for the project.

2.2.1 STATION AND DISTANCE

Historically, the Department used station and distance (left & right) to locate, design and construct its highways. During preliminary surveys and through the staking of the project, all survey and design was based on an assumed coordinate system using the bearings that were derived generally from a set of as built plans. This type of survey is no longer acceptable to the Department.

2.2.2 CONTROL SURVEYS

All control surveys will be completed under the direct supervision and responsible charge of a land surveyor licensed to practice in the State of Montana. Direct supervision is defined in the Administrative Rules of Montana (ARM) Title 24, Chapter 183, Sub-Chapter 3. The land surveyor will also be in responsible charge of the survey as defined in ARM Title 24, Chapter 183, Sub-Chapter 3, and Montana Code Annotated (MCA) Title 37 Chapter 67

2.2.2.1 Control Surveys – Local Coordinate System

Control surveys based on a local coordinate system are acceptable, but in all cases, the use of local (assumed) coordinates should be restricted to small projects such as bridge relocations, safety improvements, signal and signing projects, etc. However, elevations can only be assumed if approved by the Project Manager. The Project Manager may consult with the Photogrammetry & Survey Section.

Local coordinate systems should always use an assumed coordinate large enough so that no coordinate will be less than zero (negative coordinates). The assumed values will depend on the length and direction of the project.

The preferred method to obtain the bearing source (basis of bearing) will be a solar observation. However, the land surveyor in responsible charge may consider additional methods such as previous MDT projects (as built), previous surveys (public and private) in the immediate vicinity of the project, etc. In no case will the basis of bearing be assumed.

The land surveyor in responsible charge will notify the lead bureau as to the survey mark having assumed coordinates, the basis of bearing, and the vertical datum associated with the control survey.

2.2.2.2 Control Surveys – State Plane Coordinates

The preferred coordinate system for all Department projects is state plane coordinates. As noted above, local coordinate systems may be acceptable for certain projects. The Department's ultimate goal is to use state plane coordinates exclusively. If state plane coordinates are to be used for project control, the coordinates will be relative to Montana's High Accuracy Reference Network (HARN) or the Continuously Operating Reference Stations (CORS). The published horizontal position of marks independent of the HARN, for example NGS and/or USGS triangulation marks, will not be constrained (consider as fixed control) in any project control network. However, the elevations associated with these marks may be constrained in the project control network, provided they are third order or better.

The state plane coordinate system was developed in the 1930s to provide a common, statewide reference system so that all surveys could be on the same datum or easily converted to another state's datum. The system, in theory was ideal, but was not implemented in states like Montana. The problem was not with the system but with the location of the existing control. To compute positions on state plane datum, survey ties to known control points are required. In Montana, most of the known control was located on mountaintops or in hard to reach locations, which made their use impractical. Today, with the advent of GPS, the establishment of the HARN, and CORS, this obstacle is no longer a factor.

NAD27 Coordinates. The first State plane coordinate system was developed in the 1930s based on the North American Datum of 1927 (NAD27). NAD27 used the Clarke 1866 spheroid as a reference and the grid coordinates were in US Survey feet. NAD27 coordinates, per state statute, may not be used after July 1, 1993.

NAD83 Coordinates. During the 1980's, additional, precise survey data was combined with the existing NAD27 survey data in a complete readjustment. A new datum for the geodetic reference system was selected that approximated the shape of the world, rather than just the shape of North America. This earth-centered ellipsoid is known as the Geodetic Reference System of 1980 or GRS 80. NAD83 coordinates are relative to the GRS80 ellipsoid.

Because different ellipsoids were used to develop NAD27 and NAD83, the latitude and longitude for a point computed under NAD27 are different from the latitude and longitude for the same point computed using NAD83. There is no precise way to convert from NAD27 to NAD83 because of the different ellipsoids used for each system; therefore, conversion of NAD27 coordinates to NAD83 coordinates are not acceptable for the use on Department projects.

NAD83 Single Zone. With the development of NAD83, Montana elected to develop a new state plane coordinate system. The old NAD27 system consisted of three zones and state plane coordinates were based on the U.S. survey foot. The NAD83 coordinate system consists of one zone for the entire state and the grid coordinates are based on the meter or the international foot.

The NAD83 single zone has advantages and disadvantages. The advantages are that the state is covered by a single zone instead of three zones and ground distances are larger than the corresponding ellipsoid distances. A disadvantage of a single zone is the magnitude of the scale factor. The scale factor in conjunction with the elevation factor still needs to be applied to ground distances to obtain grid distances regardless of the state plane coordinates system used.

Points of origin and lines of exact scale for NAD83 single zone are shown in Figure 2-3. Grid distances are computed from measured ground distances by applying scale factors. The final scale factor is referred to as the combination scale factor (CSF) and is the product of the scale factor and the elevation scale factor. Figure 2-4 shows the relationship between ground distances, ellipsoid distances and grid distances.

For a complete, legal definition of both the NAD27 and NAD83 systems, refer to Montana Code Annotated (MCA) for the current state statute.

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```
TH0342 ***********************************************************************
TH0342  DESIGNATION -  V 62
TH0342  PID         -  TH0342
TH0342  STATE/COUNTY-  MT/DANIELS
TH0342  USGS QUAD   -  FOUR BUTTES (1973)
TH0342
TH0342                   *CURRENT SURVEY CONTROL
TH0342  ___________________________________________________________________
TH0342* NAD 83(1986)-  48 48 33.     (N)    105 32 46.     (W)     SCALED
TH0342* NAVD 88     -       740.479  (meters)    2429.39   (feet)  ADJUSTED
TH0342  ___________________________________________________________________
TH0342  GEOID HEIGHT-        -17.34  (meters)                  GEOID03
TH0342  DYNAMIC HT  -       740.601 (meters)    2429.79  (feet)  COMP
TH0342  MODELED GRAV-     980,749.8   (mgal)                    NAVD 88
TH0342
TH0342  VERT ORDER  -  SECOND    CLASS 0
TH0342
TH0342.The horizontal coordinates were scaled from a topographic map and have
TH0342.an estimated accuracy of +/- 6 seconds.
TH0342
TH0342.The orthometric height was determined by differential leveling
TH0342.and adjusted by the National Geodetic Survey in June 1991.
TH0342
TH0342.The geoid height was determined by GEOID03.
TH0342
TH0342.The dynamic height is computed by dividing the NAVD 88
TH0342.geopotential number by the normal gravity value computed on the
TH0342.Geodetic Reference System of 1980 (GRS 80) ellipsoid at 45
TH0342.degrees latitude (g = 980.6199 gals.).
TH0342
TH0342.The modeled gravity was interpolated from observed gravity values.
TH0342
TH0342;                 North        East    Units  Estimated Accuracy
TH0342;SPC MT       -   513,970.      890,260.      MT  (+/- 180 meters Scaled)
TH0342
TH0342                    SUPERSEDED SURVEY CONTROL
TH0342
TH0342  NGVD 29 (??/??/92)  739.907  (m)        2427.51   (f) ADJ UNCH    2 0
TH0342
TH0342.Superseded values are not recommended for survey control.
TH0342.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
TH0342.See file dsdata.txt to determine how the superseded data were derived.
TH0342
TH0342_U.S. NATIONAL GRID SPATIAL ADDRESS: 13UDQ599063(NAD 83)
TH0342_MARKER: DB = BENCH MARK DISK
TH0342_SETTING: 7 = SET IN TOP OF CONCRETE MONUMENT
TH0342_STAMPING: V 62 1934
TH0342_STABILITY: C = MAY HOLD, BUT OF TYPE COMMONLY SUBJECT TO
TH0342+STABILITY: SURFACE MOTION
TH0342
TH0342  HISTORY     - Date     Condition       Report By
TH0342  HISTORY     - 1934     MONUMENTED       CGS
TH0342
TH0342                    STATION DESCRIPTION
TH0342
TH0342'DESCRIBED BY COAST AND GEODETIC SURVEY 1934
TH0342'5.9 MI W FROM SCOBEY.
TH0342'5.9 MILES WEST ALONG THE GREAT NORTHERN RAILWAY FROM SCOBEY, DANIELS
TH0342'COUNTY, 12 FEET WEST OF MILEPOST 105, AND 38 FEET NORTH OF THE
TH0342'CENTERLINE OF THE TRACK.  A STANDARD DISK, STAMPED V 62 1934 AND SET
TH0342'IN THE TOP OF A CONCRETE POST.
```

Figure 2-1
Sample NGS Data Sheet

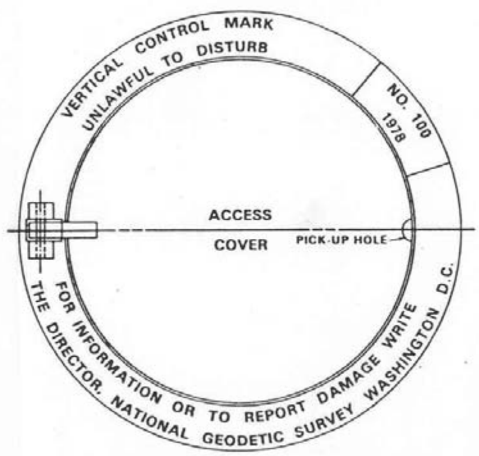
New NGS Vertical Control Marker

In 1978 the National Geodetic Survey (NGS) introduced a new, improved bench mark into the National Vertical Control Network. The reference point for the elevation is the top of a stainless steel rod. The rod is located inside a protective aluminum casement that bears the NGS logo and the stamped bench mark designation. Users can obtain access to the rod by lifting a hinged access cover.

The bench mark is designed to prevent near-surface soil movements, e.g., frost heave, shrinkage, swelling, from disturbing the mark. This is accomplished by encasing the rod in a lubricated sleeve to the depth of expected soil movement and by anchoring the rod in the soil below.

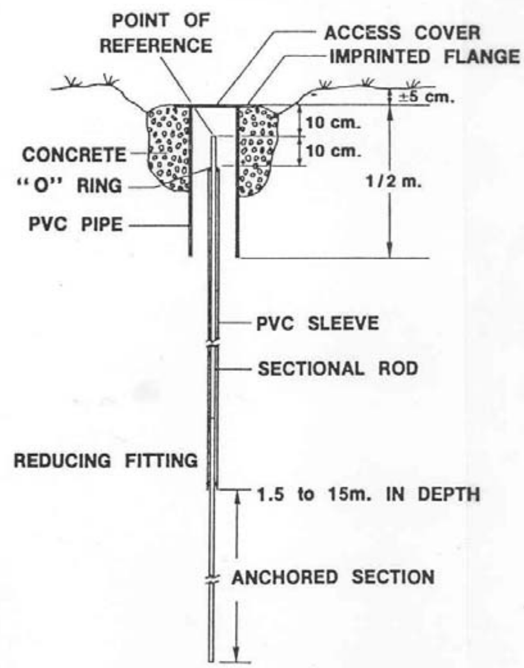
Top and side views of the bench mark are depicted below. Additional information about this mark can be obtained by writing to the Director, National Geodetic Survey, National Ocean Survey, NOAA, Rockville, Md. 20852.

TOP VIEW



BENCH MARK LOGO FLANGE & ACCESS COVER

SIDE VIEW



U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Survey

Figure 2-2
New NGS Vertical Control Mark

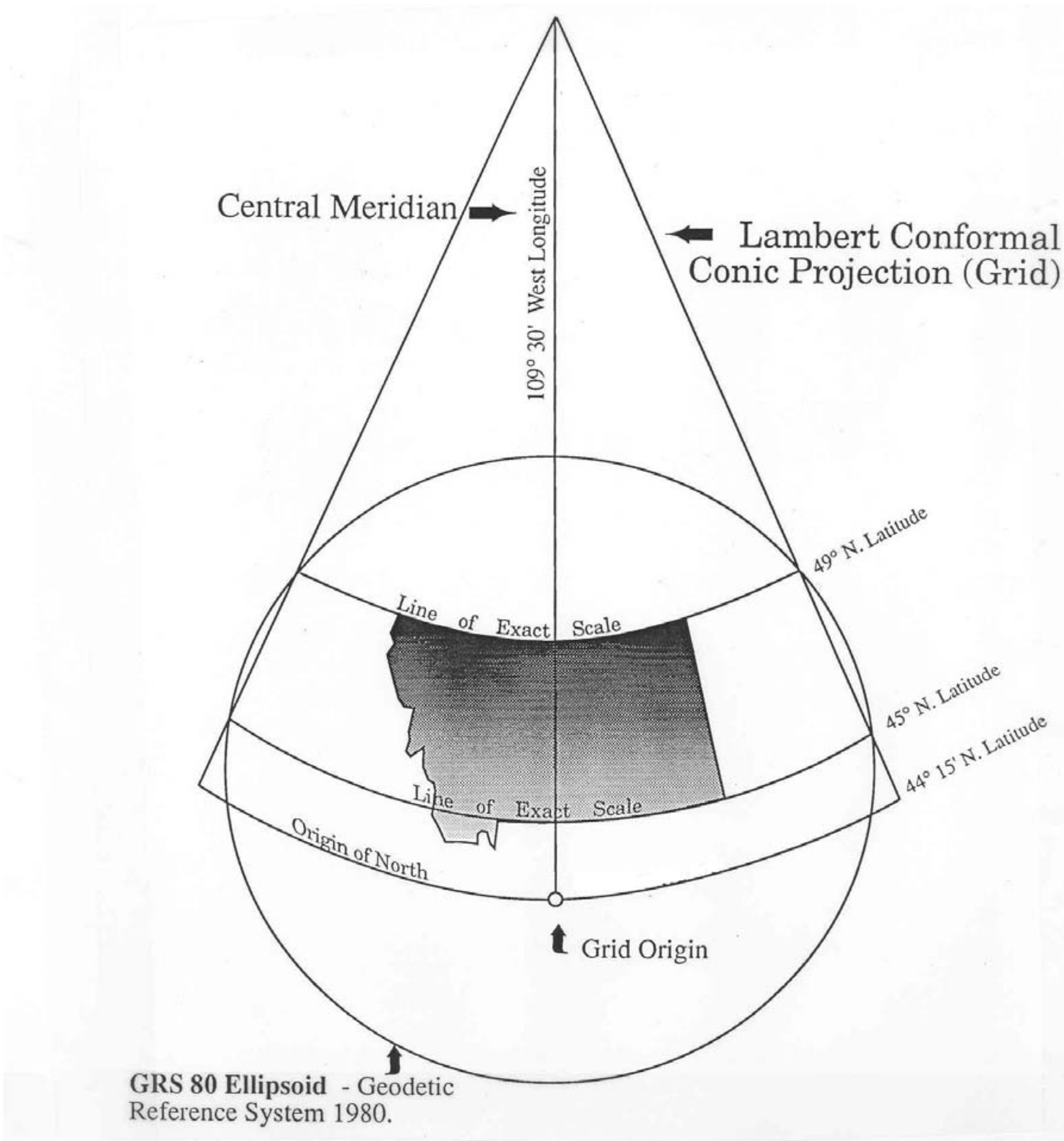
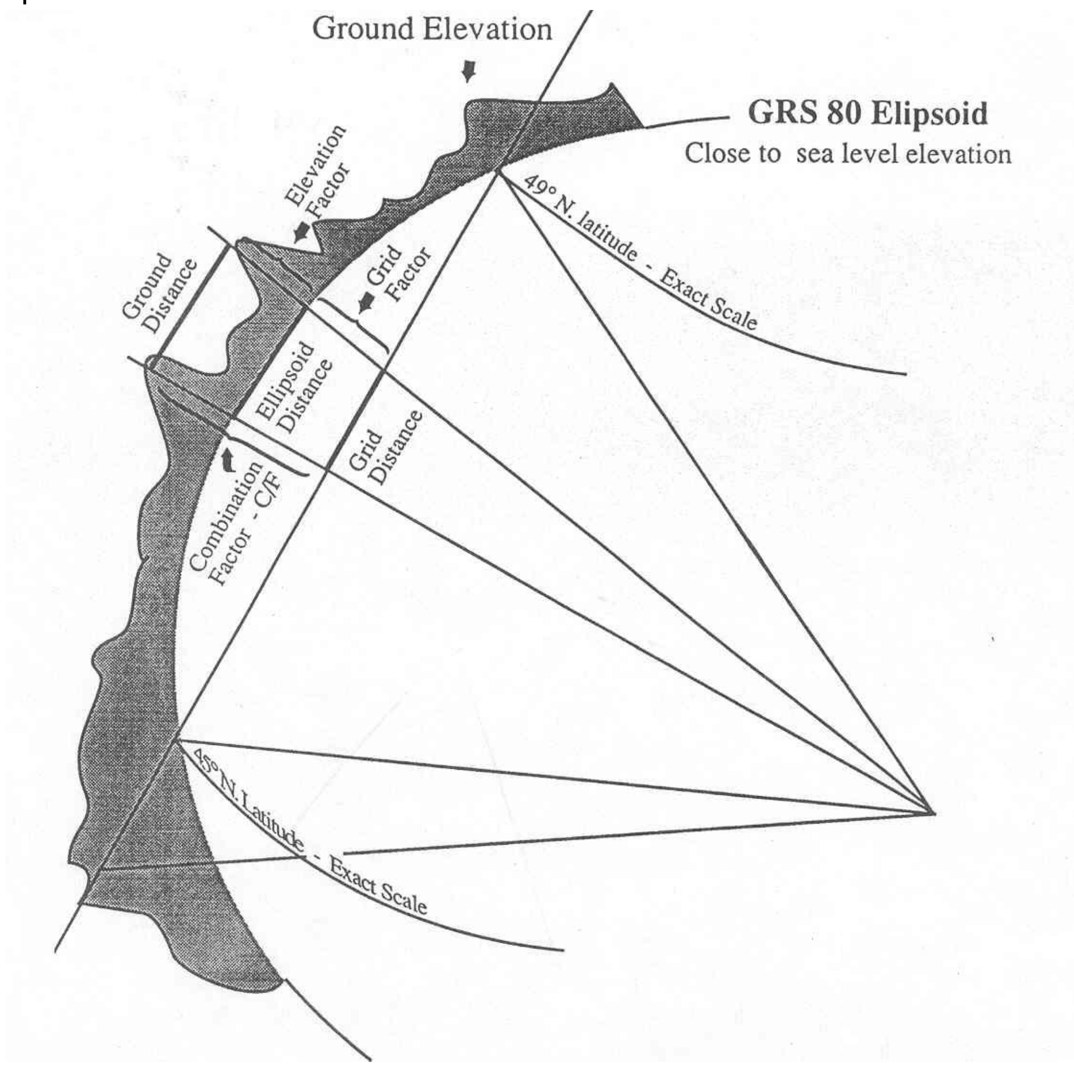


Figure 2-3
NAD83 Single Zone
Points of Origin and Lines of Exact Scale

NOTE: The grid distance is smaller than the ellipsoid distance between north latitudes 45° and 49°. Outside those latitudes, the grid distance is larger than the ellipsoid distance.



Elevation Factor-A factor used to compute the difference between ground distance and ellipsoid (sea level) distance.

Scale Factor-A factor used to compute the difference between ellipsoid (sea level) and grid distance. Always less than 1.00000 between lines of exact scale.

Combination Scale Factor (CSF)-A factor used to compute the difference between ground and grid distances. It is equal to the product of the elevation factor and the scale factor.

Figure 2-4
NAD83 Single Zone
Scale Factors

Report on Relocation of a Reset Bench Mark

Station Designation:	Level Line Number:	State:	County:
Latitude:	Longitude:		Position Accuracy: +/– ______ Scaled GPS Other: ______
Project Name:		Highway Name:	Key #

Information About Old Mark (circle or check options):

Exact Stamping of Old Disk: ______

Agency Pre-Cast in Disk/Monument Cover: ______

Published Elevation of Old Bench Mark: ______ Meters Feet Datum: ______

Old description agrees as found? Very well More or less Poorly Not at all

Old monument solidly in ground? Yes No, explain: ______

Any damage to disk or monument? No Yes, explain: ______

Anticipated date old mark to be Disturbed or Destroyed ______

Describe reason for reset: ______

Information About New Mark:

Exact Stamping of New Disk: ______ Date Set: ______

Agency Pre-Cast in Disk/Monument Cover: ______

Type of Disk Set: ______ Magnetic Material: ______

Site suitable for use with GPS geodetic surveying (e.g., few obstructions to satellites) Yes No Don't know

Setting Classification of New Monument (circle monument type 1, 2 or 3; circle or check options):

1. Concrete Post:
   a. Diameter of Monument: ______ m Depth of Monument: ______ m
   b. Top of Monument: Flush Projecting Recessed ______ m, with ground.
2. Disk Set in Drill Hole:
   a. Rock Outcrop or Boulder Approximate exposure: ______ m by ______ m
   b. Bridge Abutment or Other, explain: ______
   c. Mark relationship with surface: Flush Projecting Recessed ______ m, with ______
3. Rod Mark Driven to Refusal:
   a. Depth of rod driven: ______ m To refusal , Slow time met Depth of grease filled sleeve: ______ m
   b. Top of rod recessed ______ cm below monument cover.
   c. Top of monument cover: Flush Projecting Recessed m, with

Reported By: Date: ______

Agency: ______ Contact: ______

Address: ______ Telephone: ( ) ______

City / State / Zip: ______ Fax: ( ) ______

E-mail: ______

Form 2-1
Report on Relocation of a Reset Bench Mark

Station Description

New Station Designation:

Description of Physical Monument

To reach the station from

Permanent Station Reference Objects

Distance to Mark	Direction to Mark	Description of Reference Objects

Form 2-2
Station Description

Observations for Relocation of Bench Mark

Original Mark Stamping: ____	Replacement Mark Stamping: ____
PID (if known): ____	Date of Leveling: ____
Elevation: ____ (ft / m)	Computed Elevation:
Vertical Datum: NGVD 29 or NAVD 88	(from below) ____ (ft / m)

State: County: Latitude: N Longitude: W Hz. Datum:

Level Instrument: Manufacturer: ____ Model Number: ____ Serial Number: ____
Rod #1: Manufacturer: ____ Model Number: ____ Serial Number: ____ Rod Scale Units:
Rod # 2 (optional): ____ Model Number: ____ Serial Number: ____ Rod Scale Units:

Forward Running: Old to New (ft / m) Observer: Rodman # 1: Rodman #2:

Start Time: Temperature: F / C Sun Conditions: Wind Conditions: Stop Time:

Point	Backsight	H.I.	Foresight	Elevation	Distance	Remarks
	Starting or Published Elevation					
	Forward Running Elevation / Total Distance					

Backward Running: New to Old (ft / m) Observer: Rodman # 1: Rodman #2:

Start Time: Temperature: F / C Sun Conditions: Wind Conditions: Stop Time:

Point	Backsight	H.I.	Foresight	Elevation	Distance	Remarks
	Starting Elevation from Forward Running Elevation					
	Backward Running Elevation / Total Distance					
Forward Starting or Published Elevation						
Backward Running Elevation						
Difference = Forward Starting or Published Elevation - Backward Running						

Final New Elevation = Forward Running Elevation - (*Difference ÷ 2) *Maintain sign of Difference

Agency / Firm: ____ Submitted By: ____
Address: ____ Telephone: ( ) ____
City / State / Zip: E-mail:

Form 2-3
Observations for Relocation of Bench Mark

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Chapter 3

Surveying Equipment, Measurements and Errors

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Table of Figures and Forms

Chapter 3

Surveying Equipment, Measurements and Errors

3.1 EQUIPMENT

The procurement and maintenance of surveying equipment, tools and supplies are important parts of the Department's survey effort. Proper care in the use, storage, transportation and adjustment of the equipment is a major factor in the successful completion of a survey. Lack of good maintenance practices can jeopardize the efficiency and accuracy of the survey. This manual addresses the various types of survey equipment used by the Department's construction/survey personnel, the maintenance and care of the equipment and general procedures for surveys using the equipment. The majority of surveys done by and for the department utilize total stations, Global Positioning System (GPS), engineers levels (optical and digital) and data collectors. Appendix A includes sample notes associated with various field surveys. These sample notes may be beneficial in cases where field notes are taken and/or helpful to determine information that should be recorded in the data collector.

It is the Engineering Project Manager (EPM) and/or the party chief's responsibility to train all crew members in the proper use of surveying equipment and the maintenance of all surveying instruments, tools and supplies. The Photogrammetry & Survey Section or the District land surveyor should be contacted if additional training beyond the instruction provided by the EPM is required.

3.1.1 PERSONAL USE OF STATE ISSUED EQUIPMENT

Refer to current management memos and/or MDT policies regarding the use of state issued equipment for personal use.

3.1.2 GENERAL INSTRUMENT CARE AND SERVICING

Surveying instruments are designed and constructed to provide years of reliable use. Although they are constructed for rugged field conditions, the mechanical components and electronics of precision instruments can be damaged by careless acts or inattention to the procedures for use, care and adjustment of the instruments.

3.1.2.1 Operator's Manual

An operator's manual is furnished with each new instrument. Among other information, the manual contains basic instructions for operation of the instrument and describes recommended servicing and adjusting methods. The manual should be kept with the instrument at all times. Study the manual before using the instrument, particularly before making field adjustments. If the manual is lost, stolen or damaged beyond use, obtain a replacement copy of the manual.

3.1.2.2 Routine Care of Equipment

Before making the first set-up of the day, visually inspect the instrument for damage. Check the machined surfaces and the polished faces of the lenses and mirrors. Try the clamps and motions for smooth operation (absence of binding or gritty sound).

Clean the exterior of the instrument frequently. Any accumulation of dirt and dust can scratch the machined or polished surfaces and cause friction or sticking in the motions. Remove dirt and dust with a clean, soft cloth or with a camel-hair brush. Clean non-optical parts with a soft cloth or clean chamois.

Clean the external surfaces of lenses with a fine lens brush and, if necessary, use a dry lens tissue. Do not use silicone-treated tissues because they can damage coated optics. The lens may be moistened before wiping it, but do not use liquids (oil, benzene, etc.) for cleaning. Do not loosen or attempt to clean the internal surfaces of any lens.

After an instrument has been used in damp or cold situations, use special precautions to prevent condensation of moisture inside the instrument. If the instrument is used in cold weather, leave it in the carrying case in the vehicle during non-working periods rather than take it into a heated room. If you store the instrument in a heated room overnight, remove it from the carrying case. If the instrument is wet or frost-covered, bring it into a warm, dry room, remove it from its case and leave it at room temperature to dry out.

3.1.2.3 Vehicular Transport

Transport and store instruments in positions that are consistent with the carrying case design. For example, total stations should be carried and stored in their correct position. Many instrument cases indicate the position in which they should be transported.

Treat tribrachs, prisms and tripods with care. Carry them in their shipping cases or cushion them with firm polyfoam or excelsior-filled cases to protect them from jolting or vibrating excessively.

3.1.2.4 Casing and Uncasing

Before removing an instrument, study the way it is placed and secured in the case. Place it in the same position when you return it to the case. In removing the instrument from the case, carefully grip it with both hands, but do not grip the vertical circle standard or where pressure will be exerted on tubular or circular level vials.

3.1.2.5 Setups

Whenever possible, the instrument should be used in areas where operation is not dangerous to the instrument operator or the instrument. Select stable ground for the tripod feet. Do not set an instrument in front of or behind a vehicle or equipment that is likely to move.

In cold or hot weather when vehicle climate controls are used, survey instruments should be acclimated to outside conditions for an adequate period of time prior to final setup adjustments.

At the survey mark, firmly set the tripod with its legs spread wide. Push along the legs, not vertically downward. Extra precautions should be taken on smooth surfaces. The total station should **not** be attached to the tripod.

Always have the tripod firmly set before removing the instrument from its carrying case. Immediately secure the instrument to the tripod. If a total station is to be used, remove the instrument from the tribrach. Center and level the instrument over the mark using only the tribrach. Then place the total station in the tribrach for final leveling and verification that the instrument is still centered above the mark.

Never leave an instrument or its tribrach on the tripod without securing either to the tripod. Moderate pressure on the fastener screw is sufficient. Excessive tightening causes undue pressure on the foot screws and on the tribrach spring plate. Make sure the tribrach clamp is in the lock position.

3.1.2.6 Field Adjustments

Frequently check level vials, optical plummets, tripods, etc. for proper adjustment. In the field, make adjustments only when the instrument results are poor or require excessive manipulation.

Instruments should only be checked under favorable conditions. Only the adjustments described in the manual for the instrument should be made in the field or shop. Do not "field strip" (dismantle) instruments.

3.1.2.7 Major Adjustments

When an instrument has been damaged or otherwise requires major adjustments, contact the Construction Bureau. Indicate the type of repairs needed. In the case of total stations, digital levels or optical levels, describe the conditions under which the instrument does not function properly. Indicate if a "loaner" instrument is required.

3.1.3 EQUIPMENT DESCRIPTIONS

Specific surveying equipment is described below, along with its uses and any special precautions for its care.

3.1.3.1 Total Stations

A total station is used for measuring both horizontal and zenith angles as well as slope distances. In addition, they also have features for measurement to points that cannot be directly observed (offset measurement) and basic Coordinate Geometry (COGO). At one time, total stations were classified as either directional or repeating instruments. Most total stations have the ability to make horizontal angular measurements using either the directional method or the repetition method.

Directional Method The horizontal circle remains fixed during a series of observations. The direction of each foresight is measured in relationship to the backsight. The mean horizontal angle is then equal to the average of all the individual angles.

Repetition Method Successive measurements of an angle can be accumulated. The mean angle is then equal to the sum of the total angle divided by the number of observations.

Procedures The directional method will be used exclusively for the control survey, ties to aerial photography control points (targets), property corners, right of way, property controlling corners and secondary control traverses. All horizontal angles will be measured clockwise (angle right) from the backsight regardless of the size of the angle.

Special Care Although total stations are ruggedly built, careless or rough use and unnecessary exposure to the elements can seriously damage the instruments. If they are handled reasonably, the instruments will provide consistently good results with a minimum of down time for repair or adjustment. Some general guidelines for the care of instruments are:

- Transport and store instruments in positions that are consistent with their carrying case design. Protect the instruments from excessive vibrations by carrying them in their shipping cases.
- Instruments should be removed from the case with both hands. Generally, instruments are equipped with a carrying handle; use one hand to grip the handle and the other to support the base. Use one hand to continually support the instrument until the tribrach lock is engaged or the tripod fixing screw is secured.
- In most cases, total stations and other instruments should be removed and re-cased for transportation to a new point. If the instrument has a carrying handle, you can use the handle for walking the instrument between set-ups; however, it is recommended to case the instrument for transportation.
- The instrument should not be placed on the ground since dust or dirt can accumulate on the threads and the base plate.
- As feasible, protect the instrument from moisture.
- Never carry the instrument on the tripod.
- Turn the instrument off prior to removing the battery.
- Remove the battery from the instrument before the instrument is placed in its carrying case.
- Never use a total station for a solar observation unless an approved solar filter is used. This will destroy an element in the EDM, plus damaging the eye of the observer.

3.1.3.2 Global Positioning System Instruments

The Department uses Global Positioning System (GPS) receivers for various types of surveys and data collection. GPS receivers may be classified as hand held, mapping grade and survey grade receivers. Regardless of the type of GPS receiver, all final horizontal positions (latitude and longitude and/or state plane coordinates) of the observed marks will be relative to the North American Datum of 1983 (NAD83) and not the North America Datum of 1927 (NAD27) or the World Geodetic System of 1984 (WGS84).

Hand Held Receivers The less expensive GPS receivers obtain only limited information from the satellites. This type of receiver can be obtained from sporting good stores and other retailers. They are typically small, portable, battery powered and have a built in display. Currently the expected point positioning accuracy with selective availability disabled is approximately 30 ft (10m) horizontal. Typical uses are:

- to search for NGS bench marks
- to search for property corners and/or property controlling corners
- wetland delineations

Mapping Grade Receivers These receivers are generally used to export the collected data to an external databases such as Geographical Information System (GIS). Besides obtaining point positions, they also have an advantage over the hand held receivers since the data collected can be differentially corrected. This technique requires two receivers. One receiver is referred to as the base and is located on a known position. The second receiver is referred to as the roving receiver and it is placed over the point(s) to be positioned. Common satellite data is then stored in the base and the rover receivers. In the office, the satellite data is processed to compensate for position errors at the marks occupied by the rover. Expected horizontal accuracy can be as good as 3 ft (1m) typical uses are:

- marking locations of such things as roadway images (MDT’s Photolog System)
- boring/core hole locations
- wetland delineations
- sand pits, stock piles, rest areas, Remote Weather Information Sites (RWIS), etc
- road geometrics

Survey Grade Receivers Are single or dual frequency. Information obtained is generally post processed to arrive at positions of the occupied points. These receivers may also have the ability to perform Real Time Kinematic (RTK) surveys. Only dual frequency receivers will be used to observe base lines in excess of 6.2 miles (10km). Geodetic antennas having a ground plane are required in some cases. Expected horizontal accuracies can be as good as 0.1 ft (0.03m). Typical uses are:

- HARN densification (post processed static/fast static)
- project control (post processed static/fast static)
- cadastral surveys (post processed fast static and/or kinematic and/or RTK)
- project control densification (post processed fast static and/or kinematic and/or RTK)

Precautions Prior to commencing a GPS project insure that the latest versions of all software have been obtained, all tribrachs have been adjusted as outlined below and all cables and connections have been visually checked.

3.1.3.3 Tribrachs

A tribrach is the detachable base of all total stations, and they are also used to attach prisms to a tripod. A Department tribrach is equipped with a bull's-eye bubble (circular level) and optical plummet.

Special Care The tribrach is an integral part of the precision equipment and should be handled accordingly. It should be transported in a separate compartment or other container to prevent damage to the base surfaces, bull's-eye level and optical plummet eyepiece. Over-tightening of the tripod fastener screw can put undue pressure on the leveling plate.

Adjustments An out-of-adjustment tribrach will cause centering errors. Each tribrach should be routinely checked for centering. Using a plumb bob is quick method for checking if the tribrach is out of adjustment. To perform this task, center the instrument over the point using the plumb bob, remove the plumb bob and check the centering using the optical plummet. If the error exceeds 0.01 ft (0.003m) use one of the following methods to correct the centering error.

One field method used to adjust for centering errors is to mark and rotate the tribrach 120 degrees at a time on a tripod. Before adjusting the optical plummet, adjust the bull's-eye bubble by using the instrument plate level bubble. For the first sighting, draw a line with a soft pencil on the tribrach head around the tribrach base. Carefully level the tribrach and mark the sighting point on the ground using the optical plummet. Then rotate the tribrach 120 degrees, carefully set it in the pencil marks, re-level it and mark the new sighting point. Then rotate a third time and repeat the procedure. If the tribrach is out of adjustment, the three rotational marks should form a triangle. Adjust the optical plummet to the center of the triangle using the capstan screws. Repeat the test to verify the adjustment triangle is minimized.

A tribrach-adjusting ring is the preferred method. Place a tribrach on a tripod and the adjusting ring in the tribrach. Place the tribrach to be adjusted upside down on the ring. Look through the optical plummet and pick out a well-defined point on the ceiling. Turn the leveling screw on the bottom tribrach to center the optical plummet on the selected point on the ceiling. Rotate the top tribrach on the ring 180 degrees. If the cross hair stays on the point when rotated, the optical plummet is in adjustment. If not, use the leveling screws on the bottom tribrach to eliminate one half of the error. Eliminate the remaining error with the adjusting screws on the optical plummet. Repeat the procedure until the cross hair rotates on the point. The tribrach does not have to be level to perform the adjustment.

When adjusting the optical plummet, slightly loosen the appropriate capstan screw and equally tighten the opposite capstan screw. Use caution when tightening the capstan screws since they can easily be twisted off. Refer to the instrument user manual for detailed instructions.

3.1.3.4 Electronic Distance Measuring Instruments (Total Stations)

The development of electronic distance measuring instruments (EDM/EDMIs) has had a profound effect on the surveying profession. Linear measurement, in any practical range, can be made speedily and accurately due to the development of these instruments. The Department no longer supports nor recommends the use of EDMs that are independent of a total station.

Most EDMs have approximately the same distance measuring accuracy when operated in accordance with the manufacturer's instructions. Every instrument has an inherent plus or minus error in every measurement plus a small error based on parts per million of the distance measured.

The primary differences among makes and models of EDMs are the distance they can measure with one or multiple reflector prisms and the time required to make a measurement. Total stations incorporate EDMs as well as provisions for angular measurements and basic coordinate geometry (COGO).

Operating and maintenance procedures are covered in the manuals supplied with each instrument.

Advantages Some of the advantages of using an EDM are:

- a reduction in the time and crew size required for most measurements
- the ability to measure across traffic or construction operations without inconvenience to others (motorists or construction crews) or undue hazard to the survey crews
- the ability to measure otherwise inaccessible points, such as across deep canyons or rivers
- the ability to set many points from a relatively sparse control network, which is especially useful in construction staking
- the ability to measure with increased precision and consistency
- the ability to quickly establish better supplemental control for construction staking
- interface with a data collector

Base Line Report In 2002 and 2003, NGS completed new observations of all calibrated base lines in Montana. NGS base line data can be obtained from NGS's web site at http://www.ngs.noaa.gov/products_services.shtml#EDMI. Once a year and prior to commencing a control survey, distance measurements obtained from the total station must be compared against an NGS calibrated base line. All measurements associated with the base line report will be metric units. A comparison consists of obtaining and recording the measurements shown on Form 3-1. A blank base line report form is included in the Appendix. This report can be copied and taken to the base line, completed in the field and then submitted. An alternative method is to obtain the required measurements and then complete the online version of the base line report that is available on the MDT web page at http://www.mdt.mt.gov/publications/forms.shtml#survey. Prior to the observations, check and adjust tribrachs and tighten all tripods and compare the thermometer and barometer against a standard. The barometer should be compared with what is known as station or absolute pressure. Larger airports will have this pressure. The barometer must be taken to the airport and adjusted to the given station pressure. Station pressure is not the pressure that is broadcast during a weather report. At the baseline, record the shaded temperature, the station pressure, instrument heights and prism height and all other required information including the measured distances. It is advisable to complete the reporting form and the analysis of the measurements *before* leaving the baseline.

Caution An incorrect barometric pressure or temperature will affect the measured distances of the longer lines. The prism offset in the instrument also needs to be set correctly. Department prisms have an offset of minus 30mm (-30mm). The sign of the offset is critical.

An analysis of the measurements consists of a comparison of the differences between the measured mark to mark distance and the NGS reported mark to mark distances. The standard errors of the instrument must be known. The standard errors are given in the operator's manual and consist of a constant error and a part per million error (refer to Form 3-1).

As an example, assume the distance from the 0m to the 150m mark shows a difference of 0.0016m. The standard errors per the instrument are 0.005m + 3 part per million (ppm). Is this difference acceptable? If the difference is less than or equal to 2 times the standard errors, the answer is yes. In this example, twice the standard deviation for this line would be equal to 2[0.005 + (3 x 150/1,000,000)] = 0.0109m. The measured difference (0.0016m) is less than 0.0109m; therefore, the difference is acceptable.

As an additional example, determine whether the difference of 0.0030m is acceptable for the line from the 0m to the 1420m mark. The computation is 2[0.005 + (3 x 1420/1,000,000)] = 0.0185m. The difference of .0030m is less than 0.0185m, so the difference is acceptable. All mean mark-to-mark differences should be acceptable. If not, repeat the test after checking the tripods, optical plummets, barometer, and thermometer. Send all base line reports to the Photogrammetry and Survey Section.

3.1.3.5 Miscellaneous Equipment

The Department uses a wide range of sights in conjunction with total stations. The main purpose of a sight is to provide a reference that is visible to the instrument operator. In this context, sights may be required for line, distance, or a combination of line and distance.

Gammon Reel and Plumb Bob The plumb bob and Gammon reel is the old standard for short-distance sighting, particularly for establishing temporary points. Steadiness of the holder can be enhanced by the use of braces or any type of framework. Various types of inexpensive string-line targets are also available.

Prism Poles Prism poles are the most common sight used by the Department. These poles are made of various materials and in different lengths. The more common prism poles can be extended to allow for changing the height of the prism to avoid obstructions.

Prism Pole/Rod Levels The circular level (bull's-eye) is used to maintain both level rods and prism poles in a vertical position. An out-of-adjustment bubble used on either a level rod or a prism pole can cause errors in both angle and distance measurements. The bubble is easily adjusted by using Hold A Pole™ or a similar device.

Force-Centering Targets/Prisms Tribrach-mounted prisms are required for the control survey. The range poles mentioned above are not to be used for the control survey nor should they be attached to a tribrach to extend the height of the prism.

Prisms Most manufacturers of EDMs supply special prisms and prism holders that are compatible with their equipment. The single lens, tiltable holder with provisions for direct connection on the top of a prism pole or attached to a tribrach is the most common type used in most survey work. The prism assembly is generally equipped with a sighting target mounted above or below the prism to provide parallel sight between the sighting and measuring beams.

The maintenance of parallel sight is more significant in the accuracy of measurements as the distance is decreased. The use of the tiltable prism assembly maintains the parallel sight relationship.

It is important that the proper prism constant is used; otherwise, a systematic error will be introduced into all the measurements made between a particular EDM and prism. The best way to verify that true measurements will be made is to test the EDM and prism on a calibration base line. The Department only uses prisms that have a manufactures specification of -30mm offset; the EDM must be set accordingly.

Compass Hand compasses are used to determine approximate directions. Directions are measured in degrees and may be a bearing or an azimuth. The Department uses a compass for obstruction diagrams associated with GPS surveys, corner search, and rough checks on the direction of a line as determined by a celestial observation. The correct magnetic declination must be used or noted.

Care and Maintenance of Equipment As with any survey equipment, proper care will extend the useful life of sighting equipment.

- Check the prism pole bubble prior to each day's use. A quick check by fixing the rod in a tripod and rotating it 180 degrees will verify the adjustment.
- Check the bull's-eye bubble on the telescoping range pole using the "Hold A Pole".
- Prism assemblies and prisms should be transported in suitable carrying cases.

3.1.3.6 Leveling Instruments

Hand Levels Hand levels are useful in level runs for quick location of turn and instrument points. They are also quite useful for elevation checks during grading operations. As with any other level, the level bubble can become out of adjustment and should be checked periodically.

Clinometers Clinometers can be used in place of a hand level and for measuring approximate angles of slopes. If the clinometer is used as a level, it should be set to zero degrees. If the clinometer is used to measure slopes, the correct scale should be utilized since they generally have two scales. The most common being degrees and percent. The degree reading is generally on the left and percent on the right. Clinometers are also used for obstruction diagrams associated with GPS surveys.

Levels (Optical and Digital) These types of levels are the standard leveling instruments used by the Department. The principle of operation is essentially the same for all types of levels. The line of sight is maintained perpendicular to the direction of gravity through a system of prisms referred to as a compensator.

These levels are fast, accurate and easy to maintain. Proper care and service are required to ensure continuous service and required precision. Do not disassemble instruments in the field. Attempt only those adjustments described in the instrument manual.

Review the previously stated guidelines for care of instruments. These guidelines are generally true for the proper care of levels, although levels may be shouldered and carried; they should be carried as vertical as possible. Additional guidelines are:

- Do not spin or bounce pendulum levels, as such movement can damage the compensator.
- Protect the level from dust. Dust or foreign matter inside the scope can cause the compensator's damping device to hang up.

- Check adjustment of the bull's-eye bubble. Make certain it remains centered when the level is rotated 180 degrees. Proper adjustment reduces the possibility of compensator hang-up.
- To check for compensator hang-up, lightly tap the telescope or lightly press on a tripod leg. If the instrument has a push-button release, use it. If the compensator is malfunctioning, send the instrument to the Construction Bureau for repair.

Care and Adjustment of Leveling Instruments Most of the comments dealing with care of instruments earlier in the chapter apply to all surveying equipment, including levels. Typically, only line of sight and bull's-eye bubble adjustments are needed for the automatic and digital levels used by the Department.

The bull's-eye bubble is adjusted first. Center the bubble and rotate the telescope 180 degrees. The bubble should remain close to the center. If not, adjust the bubble one half of the distance to the center using an adjusting pin. Center the bubble and rotate the telescope 180 degrees. Repeat the procedure until the bubble remains close to the center when the telescope is rotated 180 degrees.

The line of sight is checked next and adjusted using what commonly is known as "pegging the instrument." To adjust the line of sight of an automatic level, drive two hubs approximately 200 ft (60m) apart (hub "A" and hub "B"); drive them at a slight angle so that each has a definite high point. Set up at the midpoint between the two hubs and take readings on both hubs A and B; record the readings as "a" and "b." Then set up about 10 ft (3m) from hub B. Take readings on both hubs A and B again; record the hub B reading as "c" and the hub A reading as "d." The readings are used to determine whether the cross hair in the level needs to be adjusted. Do not move the instrument from the hub B location until you confirm that the line of sight is within tolerance:

- The true difference is a - b
- The false difference is d - c
- If a - b and d - c agree within 0.01 ft (0.003m), no adjustment is needed

If the difference is more than 0.01 ft (0.003m), the cross hair needs adjustment. The correct reading, d', is equal to d + [(a - b) - (d - c)].

For an example of pegging the instrument, assume:

- a = 4.97
- c = 5.83

- b = 5.72
- d = 5.02

Then, a - b = -0.75 and d - c = -0.81. The difference between a - b and d - c is greater than 0.01 ft, so the cross hair needs adjustment. The correct reading is d' = 5.02 + [(-0.75) - (-0.81)] = 5.08 ft. With the level still near hub B, raise the cross hair from 5.02 ft to 5.08 ft. Refer to the operator's manual for the correct procedure for adjustment of the cross hair. After making the adjustment, check it by repeating the peg method described above. The four readings may be estimated to the nearest 0.002 ft (0.001m) to keep random errors to a minimum.

The line of sight in a digital level should be adjusted using methods specific to the particular digital level being used. The operator should refer to the manual supplied with the level.

Digital Levels Differential levels associated with the control survey should use a digital level and the corresponding digital rod. It is recommended that digital levels be used for all surveys that require elevations such as additional control marks established after the control survey has been completed and photo control marks.

The digital level may be used as an optical instrument. In this case the instrument's line of sight should be verified and if required, adjusted using the "pegging method" as discussed in this section.

The Department provides their survey crews with the DiNi™ digital levels; however, equivalent digital levels are acceptable. The following outlines the suggested settings associated with the DiNi™ digital level. The instrument parameters, step VI e, must be verified and set **before** other settings are made. Additional information in a PowerPoint presentation is available on the intranet at http://www.mdt.mt.gov/publications/manuals.shtml#survey

I. RPT (Keypad #1)
   a. Number of measurements - 3 to 5
   b. Standard deviation – 0.01 ft (0.003m)

II. INV (Keypad #2)
   a. Inverted rod - NO

III. PNr (Keypad #4)
   a. Input individual point number (iPNo) or toggles to input current point number (cPNo)

b. Upper or lower case alpha and/or special characters **(if alpha selected- always use UPPER case)**

IV. REM (Keypad #5)
  a. Input point code - upper or lower case alpha and/or special characters

V. EDIT (Keypad #6) **DO NOT ATTEMPT TO EDIT OBSERVATIONS IN THE INSTRUMENT**

VI. MENU (Keypad #7)
  a. INPUT
    i. Maximum distance – 200 ft (60m)
    ii. Minimum sight – 0.5 ft (0.15m)
    iii. Maximum difference – 0.005 ft (0.0015m)
    iv. Refraction coefficient – 0.140
    v. Vertical offset – 0.00 ft (0.000m)
  b. ADJUSTMENT
    i. Select Japanese method
      1. Curvature correction - ON
      2. Refraction correction - ON
  c. DATA TRANSFER
    i. Interface 1
      1. DiNi™ - Periphery (Transfers data via HyperTerminal™)
      2. Periphery-DiNi™
      3. Set parameters
        a. Format - REC E
        b. Protocol - XON-XOFF
        c. Baud rate - 9600
        d. Parity - NONE
        e. Stop bits - 1
        f. Time out - 10 seconds
        g. Line feed - yes
        h. Name - COMP1
    ii. Interface 2 - Same as above (generally not required)
    iii. PC-demo - OFF
    iv. Update/Service – IGNORE THIS OPTION
  d. SET RECORD PARAMETERS
    i. Recording of data
      1. Remote control - OFF
      2. Record - IMEM

            3. Rod reading - R-M
            4. PNo Increment - 1
        ii. Parameter setting
            1. Format - REC E
            2. Protocol - XON-XOFF
            3. Baud rate - 9600
            4. Parity - NONE
            5. Stop bits - 1
            6. Time out - 10 seconds
            7. Line feed – YES
    e. SET INSTRUMENT PARAMETERS
        i. Height unit – FEET (METERS)
        ii. Input unit – FEET (METERS)
        iii. Display R – 0.001 ft (0.0001m)
        iv. Shut off - 10 minutes
        v. Sound – ON
        vi. Language - E 300

3.1.3.7 Tripods

Tripods provide a fixed base for all types of surveying instruments and sighting equipment. Instrument manufactures have standardized surveying tripods. The typical tripod has a 5/8-inch diameter x 11 threads per inch instrument fastener that secures the instrument or the tribrach to the tripod head. The centering range is approximately 1-1/2 inches.

Care of Tripods A stable tripod is required for precision in measuring angles. A tripod should not have any loose joints or parts that might cause instability. Some suggestions for proper tripod care are:

- Maintain firm snugness in all metal fittings, but never tighten them to the point where they will unduly compress or injure the wood, strip threads, or twist off bolts or screws.
- Tighten leg hinges only enough for each leg to just sustain its own weight when legs are spread out in their normal working position.
- Keep metal tripod shoes tight.
- Keep wooden parts of tripods well painted or varnished to reduce moisture absorption and swelling or drying out and shrinking.
- Replace the top caps on tripods when they are not in use or store the tripods such that the tops are not damaged.

- The most damage occurs to tripods when being placed into or taken out of survey vehicles. The life and usefulness of tripods can be significantly extended if compartments are constructed such that the tripod is not riding on or against other equipment.
- Wet tripods should not be stored with the leg extensions clamped.

3.1.3.8 Level Rods

Philadelphia Rod At one time the Philadelphia rod was the most widely used rod. This type of rod is made in two sliding sections held in contact by two brass sleeves. For readings seven feet or less, the back section is clamped in its normal clamped position. For greater readings, the rod is extended to its full length such that graduations on the front face of the back section are a continuation of those on the lower front strip. When extended, the rod is called a “high” rod.

Fiberglass Rod Twenty-five-foot and five-meter fiberglass rods have generally replaced the Philadelphia rod. These level rods are widely used by the Department for slope staking, bench levels and the determination of miscellaneous elevations. The added height reduces turns and its relatively short sections make it easy to transport. Although they are made of strong, resilient fiberglass, they require specific care to remain serviceable and accurate. Use the following guidelines:

- Grit and sand can freeze the locking system of the slip joints. The close fit of these joints will not tolerate foreign matter. Do not lay a fiberglass rod in sand, dust, or loose granular material.
- Lower the sections as the rod is being collapsed. Do not let them drop. Dowels through the bottom of a section keep the section above from falling inside that section. Dropping sections during collapsing will loosen the dowels and jam the telescoping. It can also shatter the fiberglass around the dowel holes.
- When the slip joint goes bad, remove the rod from service.

Fiberglass rods are *not* to be used for differential levels associated with the determination of project control marks.

Foldable Rod These rods have advantages over the fiberglass rod since there are fewer moving parts. This type of rod is recommended for differential levels associated with control marks and bench levels in lieu of the fiberglass rod.

Checking Accuracy of Level Rods An approximate check to determine if a level rod has excessive error is to extend the rod and check foot (meter) marks throughout the length of the rod with an accurate tape or chain. Do this at the beginning of control level surveys. If the rod indicates a tendency to be off, it should be checked each time it is extended. A rod that is off by 0.01 ft (0.003m) will accumulate error but can show a perfect closure when the level circuit is closed on itself.

Care and Maintenance of Level Rods Level rods should be maintained and checked as any other precision equipment. Accurate leveling is as dependent on the condition of the rods as on the condition of the levels. Reserve an old rod for rough work, such as measuring sewer inverts, water depths, etc. The care requirements common to all types of rods are:

- Protect rods from moisture, dirt, dust and abrasion.
- Clean graduated faces with a damp cloth and wipe dry. Touch graduated faces only when necessary and avoid laying the rod where the graduated face can come into contact with other tools, objects, matter or materials or may become soiled.
- Do not abuse a rod by placing it where it might fall; do not throw, drop or drag a rod, or use it as a vaulting pole.
- Keep the metal shoe clean and avoid using it to scrape foreign matter off a bench mark or other survey points.
- If possible, leave a wet rod uncovered, unenclosed and extended until it is thoroughly dry.
- Store rods either vertically (not leaning) or horizontally with at least 3-point support, in a dry place and in their protective cases.
- Periodically check all screws and hardware for snugness and operation.

3.2 MEASUREMENTS

3.2.1 ANGULAR MEASUREMENTS

Horizontal angular measurements are made between survey lines to determine relative directions of the lines. When horizontal distance measurements are applied to the derived direction, the relative position of a point is established. Horizontal angles are measured on a plane perpendicular to the vertical axis (plumb line) and always clockwise in relationship to the initial backsight. The directional method will be used exclusively for surveys associated with control traverses, ties to photo control points, cadastral surveys (right-of-way, property corners and property controlling corners) and secondary control traverses.

Zenith angular measurements are measured to determine the slope of survey lines from the horizontal plane (level line). Zenith angles are used to determine horizontal distance and vertical distances. A zero degree zenith angle is directly above the instruments

In the United States, the sexagesimal system of angular measurement is used. There are 360 degrees in the circumference of a circle. A degree is divided into 60 minutes (60'), and a minute is divided into 60 seconds (60") and decimals of a second.

3.2.1.1 Terms

The following terms are defined specifically for angular measurement as used in this manual. Their meanings may differ slightly in other contexts.

- Pointing — A pointing consists of a single sighting and reading on a single object.
- Observation — An observation is a single, unadjusted determination of the size of an angle.
- Mean — The mean (average) is the final determination of the magnitude of an angle before adjustment. At least two observations are required before a mean can be determined.
- Backsight (BS) — A backsight is a survey point that is used as an initial sight for orientation when measuring horizontal angles and directions.
- Direction — A direction is the value of a clockwise angle between a backsight and any other survey point. The readings of each backsight are reduced to zero

degrees, and the directions to the other survey points are computed from this survey point.

- Setting the Circle — Setting the circle is the act of setting a specified horizontal reading while the telescope is pointed toward a backsight. Generally zero degrees, or the calculated “back azimuth,” or a predetermined setting is used.
- Direct and Reverse Readings — A direct reading is taken with the telescope in the upright position with the vertical circle on the left (left face). A reverse reading is with the telescope inverted and the vertical circle (zenith circle) on the right (right face).
- Position — A position consists of two direct and two reverse horizontal readings (two observations). For a directional instrument, the horizontal circle remains stationary for a given position but is reset for each new position. Notes for angles turned using the directional method are grouped by position. More than one position may be required.
- Set of Repetitions — A set of repetition angles is a series of observations of the same angle. Each observation is accumulated on the horizontal circle of the instrument. Half a set is measured in the direct mode and the other half in the reverse mode.

3.2.1.2 Importance

Determination of the direction of a line, azimuth or bearing is a fundamental requirement for establishing the horizontal position of one point and its relationship with any other point in the survey.

Distance and angular measurements are of equal importance in establishing point positions. Angular errors are by far the most difficult and expensive to isolate and correct. Analysis of a traverse closure error can sometimes reveal the types of errors and aid in their elimination.

3.2.2 COORDINATE MEASUREMENTS

Most total stations have the capability to locate or stake out points using coordinates. This method may be used to stake miscellaneous items such as signs and signals. Caution should be used when using this function for various reasons, such as, it produces no written record, some total stations do not allow for consideration of a scale factor and there is a possibility of inputting incorrect coordinate values and HI’s.

3.2.3 VERTICAL MEASUREMENT

A survey may require vertical measurement in addition to linear and angular measurement. Vertical measurements establish the differences in elevation between survey points.

3.2.3.1 Importance

The determination of accurate elevations is an extremely important part of the information required for the design of highway projects. Grade lines, drainage structures and other highway features are designed in relation to existing and final elevations. Volumetric quantities are determined by preliminary (before) and final (as-constructed) cross sections or by a digital terrain model (DTM).

Due to its importance in all other phases of the project, vertical measurements to establish primary vertical control are made at an early stage in the survey. Differential levels are used to establish elevations for all project control monuments. These control monuments are then used to extend the vertical control through the project area. Subsequent level loops between the control monuments, again by differential leveling, are used to establish vertical control for photogrammetric, preliminary, construction, or additional control monuments.

3.2.3.2 Planning

By the time a project is completed from preliminary through the construction phase, each bench mark will have been used many times to provide the base for vertical measurement. Proper planning in anticipation of the future uses of vertical control points (bench marks) is as essential as that required for the horizontal control. Some considerations for the placement of horizontal and vertical points are:

- location of the primary control (project control monuments)
- permanence (outside of anticipated construction limits — do not set in fence lines, next to trees or buildings)
- accessibility (on the right of way or other accessible lands)
- type of monument set — concreted monuments (permanent), rebars (semi-permanent) and wooden hubs (temporary)

- bench mark spacing is generally at 1,000 ft (250m). Keep in mind that the horizontal control points can be used as bench marks — in other words, projects may not require the traditional bench marks if all control points have elevations
- estimated final design grade
- visibility

3.2.3.3 Methods

Vertical measurements are made directly or indirectly. The choice of the method and its procedures depends on present and future accuracy requirements and the relative cost. Consider these items in selecting the method and procedures:

- the precision of the survey should be compatible with the accuracy of the controlling monuments
- the type of equipment available
- the future survey needs

Direct Vertical Measurement This method refers to the direct reading of elevations or vertical distances. Direct elevations can be determined using altimeters, differential levels and profile levels.

Indirect Vertical Measurement Indirect vertical measurements require the use of calculations to determine elevations or vertical distances. Trigonometric levels are one example.

Prior to the development of total stations, almost all vertical measurements on highway projects were made by differential leveling. Trigonometric measurements using total stations can be a cost effective method of making vertical measurements, but some precautions need to be taken.

3.2.3.4 Differential Leveling

Equipment The standard instruments for all differential leveling are the optical or the digital levels.

The Department primarily uses fiberglass or foldable leveling rods. Each type of rod has its particular advantage under certain field conditions. Any rod used should be clean, “tight,” and have properly indexed scales. Check slip-joint rods for index periodically.

Instrument Setups The following guidelines pertain to instrument setups:

- Do not waste time by deeply embedding tripod feet. Settlement is usually insignificant. Avoid setups on hot pavement or in spongy or muddy soil.
- Set turning points so that the backsights and foresights for each setup are approximately equal. This compensates for curvature and refraction and for systematic errors in the instrument.
- Use sight distances that best fit the terrain and are the most comfortable for the instrument operator. Sight distances should not exceed 200 ft (60m).
- In steep terrain, place “turns” and instrument setups so they follow parallel paths (not along the same line).
- Take an extra turn rather than try to read the bottom or top of the rod.
- Periodically test the level to be certain the pendulum compensator is working. Point on a “natural” sight with the telescope over a foot screw and turn the screw back and forth or lightly tap the instrument. If the cross hair dips and returns to its original position, the compensator is working properly.

Turning Points and Bench Marks

- Set bench marks in stable, protected locations. Do not set spikes in utility poles because they may be hazardous to utility workers. Wooden stakes and hubs should be used only as temporary bench marks. The project control monuments should be used as the primary bench marks.
- Make each turn stable and with a definite high point. If a TP does not have a prominent point, mark the exact point with keel or paint.
- All bench marks should be described as to type (aluminum cap, red plastic cap, Morasse™ cap and rebar, etc) and include its location.

Rod Reading

- Eliminate parallax before any readings are made.
- Do not deliberate over readings. Read and call them out in a moderate rhythm.
- Whenever possible, plumb the rod with a rod bubble. In the absence of a rod bubble, slowly rock the rod toward and away from the instrument. The observer reads and records the lowest reading. The rod must be set on a sharp or

rounded projection; otherwise, the rod will rise as it is rocked and will result in a false reading.

- Avoid low, ground-skimming shots where refraction might become pronounced. Also, avoid sighting close to obstructions that might diffract the line of sight — if possible, no closer than 1 ft from an obstruction.
- Read and record to the nearest 0.01 ft (0.002m).
- Record exact splits always high or low.

3.2.3.5 Single-Wire Levels

Single-wire leveling is the most common and widely used method of vertical measurement. With proper attention to procedures, third-order accuracy may be achieved with single-wire leveling. Some of the methods will be described in general terms only. Detailed procedures may be found in publications such as those listed in Appendix B. Sample notes showing a level loop associated with a control traverse is shown in Appendix A.

Normally, single-wire notes are reduced to height of instrument (HIs) and elevations as the survey progresses. To check the elevations of bench marks (BMs) that are turned through, differences in elevations are calculated. In the case of a level loop, the algebraic sum of the backsight and foresight should equal the ending elevation of a BM minus the starting elevation of a BM. See the sample notes in Appendix A for examples. Side shots will not be used to establish any temporary bench marks (TBMs), BMs, elevations for photo control, or project control monuments.

Differential levels can be used to establish new elevations between previously established vertical control points. If the vertical error is within tolerances then a return is not required. Caution is necessary if differential levels are used to establish new elevations and the return is made to the starting vertical control point.

3.2.3.6 Double-Turn Point Leveling

This technique uses two parallel, independent foresight and backsight turn points (TPs) for each HI. Each pair of TPs is set, if possible, at an appreciable difference in elevation (preferably one-half foot or more). They are also set a few feet apart so the level will have to be rotated slightly between the two rod readings.

From each set-up, single-wire plus shots are read on both backsight TPs; minus shots are read on both foresight TPs. Notes are kept separately for each line of levels. The adjusted elevations from the two lines of TPs are averaged.

The system has some advantages in that the HI is determined from each of the two lines and misreading or misleveling blunders can be isolated immediately. However, the system is time-consuming and both lines are run in the same direction, which may not cancel systematic errors. There is also a danger in misreading the initial backsight and the final foresight. Therefore, this method should only be utilized in unique situations.

3.2.3.7 Three-Wire Leveling

With this leveling technique, the cross hair and both stadia hairs are recorded. Stadia intercepts of plus and minus shots are accumulated. The running totals are constantly monitored so balance can be maintained between totals of foresight and backsight distances. The backsights and foresights should be balanced within 32 ft (10m). Sample notes are included in Appendix A.

This technique is generally preferred over previously described methods when elevations that are more accurate are required. With the advent of digital levels, there are few reasons for the Department to use three-wire leveling. Its special requirements are below.

Instruments The Department uses levels with either a stadia ratio of 0.3 to 100 or 1.0 to 100. For precise levels, the 0.3 to 100 ratio is preferred because the stadia hairs are nearer the optical center. It also permits a greater elevation difference between the level and the TP while keeping all three cross hairs on the rod.

Most of the Department's instruments are equipped with the 1.0 to 100 stadia ratio and their use is acceptable. Spacing of the setups and TPs will need to be adjusted for the instrument used.

Rods The Department does not have invar rods suitable for three-wire levels. It is recommended that the best rod available be used on these surveys. Even new rods should be checked upon delivery, as some rods have been found to be off.

Instrument and Rod Checks Before starting each day's run, test the level for collimation error (pegging the instrument). Test at or near the first setup of the day and record the process in the field notes. If the error exceeds 0.01 ft in 200 ft (0.003m in 60m), the instrument should be adjusted. Any time the instrument has a severe jolt or bump, the instrument should be pegged.

Check the rod in the raised position to ensure there is no index difference above and below the slip joint. Recheck the rod each time it is extended.

Setups Keep all sights within 200 ft (60m). When rejected readings average more than two in every ten, shorten the sighting distance.

Turning Points and Bench Marks Railroad spikes, boat spikes, wooden stakes or hubs may be used for TPs. Permanent BMs should be numbered so they can be identified when recovered.

Bench marks can be set prior to or during leveling. Check all found monuments that are to be incorporated in the level line for stability.

Rod Readings Plumb the rod with an accurate rod level. Do not use the wave method for three-wire levels. Start the rod reading with the top stadia wire, and progress to the bottom wire. Estimate readings to the nearest 0.003 foot (0.001m). Read at moderate speed, without deliberations. Do not turn or pick up the instrument until the note keeper has verified the spread. If the spread between top and center wire and bottom and center wire exceeds 0.005 ft (0.002m), re-read all three wires. The original readings should be crossed out and new readings entered on the next line. In some cases, it may not be necessary to provide all checks as shown in the sample notes.

Level Line (Loop) The highest order of accuracy required can normally be met by a single run of three-wire levels using an optical level or a single run using a digital level between existing NGS benchmarks or project control marks. If the levels fail to close within the tolerance specified, one of several problems may exist, including but not limited to the following:

- There may be a discrepancy between the elevations of the beginning and closing bench marks. Be sure that the data sheet is the most current for the two bench marks. (There is a possibility that one or both of the benchmark elevations have been readjusted.) If the latest information was used, carry the survey to the next NGS benchmark.
- The benchmarks may be on different lines.
- You are mixing NGS and USGS bench marks (NGVD29 and NAVD88).
- You made a reading or calculation error.
- One of the two bench marks have been disturbed.

If the possibilities above do not account for the discrepancy, the run should be re-leveled in the opposite direction.

3.2.3.8 Trigonometric Vertical Measurement

Trigonometric vertical measurement is a procedure in which vertical differences in elevation are computed from slope distance and zenith angle measurements. HIs and height of sights (HSs) are required.

The development and continuing refinement of total stations are making rapid changes in the use of trigonometric vertical measurements. These instruments are of varied vertical angle accuracy and certain procedural restrictions must be applied.

Use Trigonometric levels may be used in certain cases to establish elevations in rolling to steep terrain. They are also useful for other types of surveys. Some of these are:

- check levels for long differential lines
- establishing low order bench marks by precise vertical traversing (when accuracy is difficult to attain at reasonable expense by differential leveling)
- slope staking
- cross sectioning
- topographic surveys in conjunction with total stations and data collectors
- in certain situations, photogrammetry control marks
- observations (reciprocal) to check GPS derived elevations

Accuracy Attainable The accuracy that can be obtained from trigonometric levels depends on the individual accuracies of the following:

- error in the measurement of the HI and HS
- amount of curvature and refraction in the distance measured
- error in the slope distance (minimal source of error)

Differential Versus Trigonometric Levels Differential leveling is the preferred method for obtaining elevations. Differential levels are used exclusively for the establishment of bench marks and elevations of project control marks (conventional control surveys). In some cases, trigonometric levels may be used to obtain elevations of photo control points. Trigonometric levels are generally used to obtain elevations of miscellaneous points that will be used only for the collection of topography.

3.2.4 LINEAR MEASUREMENT WITH TAPES

Surveyor's tapes are available in various lengths, of different materials and with many methods of graduations. Although the use of total stations has replaced tapes for long measurements, every crew should have a metallic and non-metallic tape available.

3.2.4.1 Taping

Use the following guidelines when taping:

- Do not become careless when plumbing over a point.
- Keep the correct tension on the tape. Tapes of 100 ft or less require 10 pounds of tension when supported along their entire lengths. Light steel tapes stretch 0.01 to 0.02 ft in 100 ft if the tension is increased to 20 pounds.
- Avoid sag by applying the proper tension to the tape.
- Apply the appropriate temperature correction factors for the taped measurements. Tapes are calibrated at 68° F. They expand or contract 0.0000065 ft per ft for each degree of change above or below 68 degrees.
- Ensure that the tape is the correct length.

3.2.4.2 Care and Maintenance of Tapes

Tape reels for metallic or fiberglass tapes save time and help prevent damage to the tape, particularly if used in construction or heavy traffic areas.

Routine care extends tape life. The following are basic guidelines for care of metallic tapes:

- After the day's work, clean and dry tapes that are soiled. In wet weather, lightly oil and then dry tapes before storing. Avoid storing in damp places.

- Clean rusty tapes with fine steel wool and cleaning solvent or kerosene. Use soap and water when tapes are dirty or muddy. To prevent rust after cleaning, oil lightly and then dry the tapes.
- Use chaining clamps to avoid kinking the tape.
- Do not place a tape where it can be stepped on or run over, unless the tape is flat, taut and fully supported on a smooth surface.
- Leave the tape on the reel unless you are making a series of pulls that are full-tape lengths. Do not wind tapes excessively tight on their reels, as it can cause unwanted stresses.
- Keep the tape straight when in use. When dragging a tape between points, watch carefully for loops forming. When pulling a slack tape, a loop can develop into a kink and easily break the tape, especially when going down hill.

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3.3 ERRORS, CORRECTIONS AND PRECAUTIONS

Factors that might influence the occurrence of errors can be roughly divided into five classes: instrument, personal, natural, random and systematic. The first three types of errors are covered below. Random and systematic errors are addressed in Chapter 4.

3.3.1 INSTRUMENT ERRORS

Make adjustments at regular intervals and particularly before starting work on a control survey. Make the adjustments under the most ideal conditions available, normally in the highway yard or shop on an overcast day. Instruments requiring major adjustments should be serviced at an authorized repair shop.

3.3.1.1 Collimation

Collimation errors associated with total stations may be determined by following the procedure outlined in the user's manual. If either horizontal or vertical collimation errors are found to be excessive, these errors should be eliminated by following proper procedures or request the Construction Bureau arrange for adjustment at an authorized repair center. Horizontal collimation errors are compensated for if a position is turned (two direct and two reverse observations on the back sight and the corresponding foresight). Vertical collimation errors are important if only single face observations are utilized, as is the case for most topographic surveys using the total station and data collector. Excessive vertical error will affect elevations of all observed topographic points.

3.3.1.2 Plate Bubbles, Bull's Eye Bubble and Optical Plummet

Normal measuring procedures do not compensate for maladjustment of either the plate bubble(s) or the optical plummet. These components must be checked more frequently than others.

Check the optical plummets prior to commencing a control survey. Check the plate bubbles routinely on each setup. If you detect error, adjust the bubbles for the mean of the error. The bull's eye bubble should be adjusted after the plate bubble. Refer to the manual supplied with the instrument.

3.3.1.3 Parallax

Parallax occurs when the focal point of the eyepiece does not coincide with the plane of the cross hairs. The condition varies for each observer because the focal length depends in part on the shape of the observer's eyeball. Parallax is also a major concern in the optical plummet.

Check for parallax every time you begin to operate a new instrument or one that has been operated by someone else. Check the optical plummet on every setup, particularly if the HI is significantly different from the last setup.

To check for parallax in the telescope, focus the telescope on some well-defined distant object. Slowly move the head back and forth, about an inch from the eyepiece, while watching the relationship of the object to the cross hairs. If the object appears to move, parallax exists.

Parallax associated with the optical plummet can be checked in a similar manner to that of the telescope.

To eliminate parallax, rotate the knurled eyepiece ring until the cross hairs are the thickest and blackest, refocus and check for parallax as described earlier. If parallax still exists, repeat the procedure.

3.3.2 PERSONAL ERRORS

3.3.2.1 Error in the Measurement of the HI and HS.

It is important that these errors do not occur. If either the HI or the HS is measured incorrectly, this error may not be detected. Error in the measurement of the HI of a total station will result in an incorrect elevation of all observed points from that setup. Measuring and recording of the HI in both feet and meters is advisable. An error in the HS of an individual foresight point will affect the elevation of only that point.

3.3.2.2 Setting Up the Instrument

Use the guidelines below for setting up an instrument:

- Be sure the tripod is in good condition and all hardware is snugly fitted.

- Push the tripod shoes firmly into the ground. Do not stamp on the tripod feet. Pressure should be parallel to each leg.
- Place the legs in positions that will require a minimum of walking around the setup. In windy conditions, additional stability can be achieved if one leg is set downwind.
- On hillsides, it is advisable to set one leg uphill and the other two legs downhill.
- If the ground is soft or muddy, drive hubs into the ground to support the tripod legs.
- Be sure the instrument is set properly over the point.
- Check the optical plummet after the instrument is set up and just before moving to another point. If the instrument has moved, check the angle just measured.
- Recheck the instrument level. The bubble should remain within one graduation when the instrument is smoothly turned through one circle (if the instrument is shaded).

3.3.2.3 Setting Sights

When tribrach-mounted prisms are used, take the same precautions as when setting up an instrument. With this equipment, “forced centering” between the prisms and the total station (and vice versa) greatly decreases the effects of plumbing errors in traverse closures. Forced centering is especially beneficial in short-course traverses. Forced centering is the traverse procedure whereby backsight, instrument point and foresight are “leap-frogged.”

Once a tribrach is set over a point, it stays mounted on the tripod over that point for all uses. The instrument and sights are transferred from point to point without disturbing the tribrach setup. This is standard procedure for the control traverse.

Use these guidelines for setting sights:

- Always check a sight before picking up to see that it has not moved.
- When setting a pole sight, plumb it with a precision equal to that required for the total survey. A twenty-second error results from a sight that is one-tenth out-of-plumb and 1,000 ft away.
- If a sight is set near ground level, check the line of sight for obstructions or for excessive heat waves. On very warm days, ground-level sights are not advisable.

3.3.2.4 Pointing

Tangent Screw Use When moving onto a target, always make the last turn of the tangent screw clockwise. Clockwise movement increases the tension on the loading springs. A final turn counter-clockwise releases tension and the spring can hang up, causing a “backlash” error.

Cross Hair Use Sight each object with the same part of the cross hair, preferably near the center of the field of view. This practice minimizes small residual adjustment errors.

The human eye can estimate the center of a wide object more accurately than it can line up two objects. For this reason, different pointing techniques should be used depending on the type and apparent size of the sight in the telescope.

When pointing on narrow sights, such as the center of a red and white target or distant range pole, straddle the sight with the double cross hairs. When pointing on wide sights, such as a lath or range pole at close range, split the sight with the single cross hair.

False Sights Reflective surfaces, such as vehicle headlights, mirrors, windowpanes, etc., can cause false readings, as well as multiple prisms along the line of sight. Always recheck measurements that could be reflected from such reflector surfaces.

3.3.2.5 Measuring Angles

Measure angles as rapidly as comfortably possible with a uniform rhythm. Take the first reading at an object, rather than fidgeting with the tangent screw trying to improve the pointing. Too much pointing time increases the probability of error through instrument settlement or atmospheric changes. Speed; however, should not be cultivated at the expense of good results. Accuracy is more important than speed.

3.3.2.6 Readings

If field notes are being taken, carefully read and call out each reading to the recorder. Call out the entire reading each time so any large blunders are caught. Have the recorder repeat the reading to the instrument operator after it is recorded.

3.3.2.7 Analyzing Field Notes

Many recording errors and inconsistencies can be caught by carefully analyzing the observations. These items should be checked in the field by the recorder and then rechecked in the office. Examples are:

- Spreads between the seconds of direct and reverse readings should be consistent and in the same direction throughout the set.
- The sums of the direct and reverse zenith angles should be consistent.
- When the direct and reverse observations of a position are in different minutes, be sure the average second value is coupled with the correct minutes value.

3.3.3 NATURAL ERRORS

3.3.3.1 Differential Temperatures

Bright sunlight striking certain parts of the instrument may cause differential expansion of the metal components of the instrument, resulting in small errors.

3.3.3.2 Heat Waves

On a hot day, heat waves can cause distortion of lines of sight near a reflecting surface. Control traverses should be halted if excessive heat waves are present.

3.3.3.3 Phase

If a sight is not evenly lighted on both sides, the instrument operator tends to point toward one side. This phenomenon, called “phase,” can be reduced by using a target with a flat surface pointed directly toward the instrument. The targets attached to the prism are useful in reducing phase.

3.3.3.4 Refraction

When light waves pass from a medium of one density into a medium of a different density, the rays change in direction (bend). The change in direction is called refraction. Because sight lines are light rays, they are refracted, or bent, by changes in the atmosphere, causing small errors in angular measurement.

Normally, the lateral refraction is insignificant in most surveys, but its effects can be further minimized by understanding and avoiding situations producing the largest refraction of line of sight. Some of these situations are:

- When the sun shines on a barren, dark surface, the surface warms relatively quickly. This warms the air, and if calm, it produces a column of warm, light air rising from the surface. Examples are:
- dark, freshly plowed fields lying between lighter colored areas of growing crops
- clear areas between heavy forests
- large bodies of warm water between land areas
- Open valleys bordered by bluffs on either side can result in refraction. If a line must pass over a valley, set the observation points as far back from the edges of the valley as possible.
- Air tends to layer parallel to the slopes of embankments or the base of foothills.

When refraction is probable in angles to be measured, or is suspected in angles that have been measured, recon the survey area and plan station locations to avoid the problem conditions listed above.

3.3.3.5 Curvature and Refraction

Curvature corrections deal with the curved surface of the earth. Generally, curvature and refraction are grouped together. In most cases, it is assumed that refraction offsets curvature by 14 percent.

The combined effect of curvature and refraction can be eliminated only by reciprocal zenith angles. Reciprocal zenith angles consist of observations at each end of the line. Reciprocal zenith angles must be used for all photo control points if the observation is from a single control point. Curvature and refraction may be compensated by:

- enabling curvature and refraction in the total station (approximate)
- performing mathematical calculations (approximate)

Total stations should have curvature and refraction enabled to correctly calculate the vertical and horizontal distances. If a data collector or surveying software is used to generate 3D coordinates from raw data, curvature and refraction should also be enabled to correctly generate 3D coordinates. Generally, curvature and refraction has more of an effect on the vertical distance than the horizontal distance.

The amount of curvature and refraction is a function of distance and atmospheric conditions. The effect is not proportional. For example, the curvature and refraction associated with a distance of 1,000 ft is 0.026 ft, but at a distance of 3,000 ft it is 0.185 ft.

The sample notes in Appendix A (Figure A-7) indicates the required information necessary to determine the elevation of a photo control point using trigonometric levels. It is important that all the information shown in the samples be included in your field notes.

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BASE LINE USER REPORT

BASE LINE DESIGNATION:	**HELENA CBL**	
INSTRUMENT OWNER:	MDT	
CREW:	COBARRUBIAS/BAUER	
COUNTY:	LEWIS & CLARK	
DATE:	01/15/03	
WEATHER:	OVERCAST, COOL	
INSTRUMENT:		
MAKE:	SOKKIA	
MODEL:	SET 2110	
SERIAL NUMBER:	D21817	
	mm	ppm
STANDARD DEVIATION:	2	2
REFLECTOR:		
MAKE:	SOKKIA	
MODEL:		
CONSTANT:	-30mm	

MARK TO MARK

Instrument Station	0 M	0 M	0 M	150 M	150 M	430 M
Reflector Station	150 M	430 M	1070 M	430 M	1070 M	1070 M
Temperature	5c	5c	5c	5c	5c	5c
Pressure	892mbars	892mbars	892mbars	892mbars	892mbars	892mbars
PPM	25ppm	25ppm	25ppm	25ppm	25ppm	25ppm
Height of Instrument (1)	1.443	1.443	1.443	1.350	1.350	1.420
Height of Reflector (1)	1.386	1.365	1.326	1.365	1.326	1.326
DH=HI-HS	0.0570	0.0780	0.1170	-0.0150	0.0240	0.0940
Measured Distance (Five Readings)						
1	150.0060	430.0010	1070.0210	279.9990	920.0090	640.0200
2	150.0050	430.0010	1070.0170	280.0010	920.0080	640.0190
3	150.0050	430.0010	1070.0150	280.0020	920.0100	640.0190
4	150.0050	430.0020	1070.0160	279.9980	920.0100	640.0200
5	150.0030	430.0000	1070.0160	280.0020	920.0090	640.0190
Average Measured Distance	150.0048	430.0010	1070.0170	280.0004	920.0092	640.0194
Measured Distance in Feet (2)	492.142	1410.764	3510.567	918.632	3018.400	2099.803
Observed Mark to Mark Distance	150.0048	430.0010	1070.0170	280.0004	920.0092	640.0194
NGS Mark to Mark Distance (3)	150.0069	429.9974	1070.0118	279.9908	920.0050	640.0144
Difference	0.0021	0.0036	0.0052	0.0096	0.0042	0.0050
2 x Standard Deviation	0.0046	0.0057	0.0083	0.0051	0.0077	0.0066

NOTES:

(1) Make all measurements including HI and HS in meters except for one slope distance in feet.

(2) This distance is for a check only and should not be used to reduce the measured distance from mark to mark.

(3) NGS Measured 2002

Return copy to: Montana Department of Transportation, 2701 Prospect, Helena, MT 59620
Attn: Photogrammetry and Survey Section

Form 3-1
Base Line User Report

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Chapter 4
Errors and Maximum Closures

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Chapter 4
Errors and Maximum Closures

Linear, angular, and vertical measurements are the basic operations performed by the survey crew. The survey crew applies mathematics (geometry and trigonometry) to convert field measurements to the horizontal and vertical relationships needed to produce maps or plans for engineering projects.

The surveyor must become adept at making the required measurements to the degree of accuracy required. The precision to which the measurements are made and the accuracies achieved by them must fit the needs for each type of survey.

The use of common sense and development of good surveying practice in all phases of a survey cannot be over-emphasized. No manual can cover all conditions that may be encountered on the actual field survey. This manual specifies certain requirements, such as a number of repeated operations to achieve a required accuracy. If modifications of these requirements are necessary, the surveyor must use sound judgment based on the equipment being used and the field conditions encountered. The stated requirements are considered the minimum. Some field conditions — heat waves or wind, for example — may make it impossible to perform some operations to a consistent degree of accuracy.

4.1 ACCURACY AND PRECISION

Accuracy is the degree of conformity with a standard or a measure of closeness to a true value. It is distinguished from precision, which relates to the quality of the operations used to obtain the result. The standard used to determine accuracy can be:

- An exact value, such as the sum of the interior angles of a triangle is 180 degrees.
- A value of a conventional unit as defined by a physical representation, such as the meter.
- The sum of the interior angles of a traverse is equal to (N - 2) times 180 degrees.
- A survey value determined by refined methods and deemed sufficiently near the ideal or true value to be held constant for the control of dependent operations. Examples are bench marks elevations published by the NGS and horizontal coordinates associated with Montana’s High Accuracy Reference Network (HARN).

Precision is the degree of refinement in the performance of an operation or in the statement of a result. The term “precise” is also applied to methods and equipment used in attaining results of a high order of accuracy, such as invar rods or precise levels.

Generally, the accuracy of a field survey depends directly on the precision of the instrument. Although due to compensating errors, surveys with high-order closures might be attained without high-order precision, such closures are meaningless. Surveys must be performed with a precision that ensures that the desired accuracy is attained. Surveys performed to a precision that excessively exceeds the requirements are costly and should be avoided.

4.2 BLUNDERS AND ERRORS

Field observations and the resulting measurement are never exact. Any observation contains various types of errors. Often some of these errors are known and can be eliminated by applying appropriate corrections. After all known errors are eliminated, a measurement will still be in error by some unknown value. Usually, the more precision used in making the observations, the less the magnitude of the unknown error. However, a measurement is never exact, regardless of the precision of the observations.

The ultimate responsibility for providing surveys that fulfill desired accuracies remains with the survey crew. To meet this responsibility, they must understand errors, including:

- the various sources of errors
- the effect of possible errors upon each observation, each measurement, and the entire survey and
- economical procedures that will eliminate or minimize errors and result in surveys of the desired accuracies

4.2.1 BLUNDERS

A blunder, also called a mistake, is an unpredictable, human mistake. It is not an error, although a small blunder may remain undetected and have the same effect as an error. Examples of blunders are:

- transposition of numbers
- neglecting to level an instrument
- misplacing a decimal point
- misunderstanding a callout
- backsighting an incorrect turning point or control point
- not extending the level rod the full length for a high rod reading and
- Input of incorrect temperature or pressure into a total station

Blunders are caused by carelessness, misunderstanding, confusion or poor judgment. Blunders may be avoided by alertness, common sense, and good judgment.

Blunders must be detected and eliminated by using proper procedures, such as:

- making independent check observations and measurements
- checking each recorded and calculated value
- closing all surveys (horizontal and vertical) and
- recording measurements to all backsights when traversing, observing side shots and/or staking out

Blunders must be eliminated prior to correcting and adjusting a survey for errors.

4.2.2 ERRORS

The error in an observation is the difference, after blunders have been eliminated, between a measured value of a quantity and the true value of that quantity.

4.2.2.1 Types of Errors

Surveying measurements are subject to systematic and random errors.

Systematic Errors A systematic error is an error that always has the same magnitude and the same algebraic sign under the same conditions.

In most cases, systematic errors are caused by physical and natural conditions that vary in accordance with fixed mathematical or physical laws. However, some may result from the observer's tendency to react mentally and physically in the same way under similar conditions.

A systematic error of a single type is cumulative. However, several kinds of systematic errors occurring in any one measurement could compensate for each other. Some examples of systematic errors are:

- incorrect prism setting on a total station to compensate for the prism offset
- thermal contraction or expansion of a steel tape
- incorrect parts per million (ppm) setting on a total station
- curvature and refraction
- incorrect length of a level rod

Although some systematic errors are difficult to detect, survey crews must recognize the conditions that cause such errors. Once the conditions are known, the effect of some systematic errors can be eliminated or minimized as follows:

- differential levels — balancing foresight and backsight distances
- standardized tapes — making appropriate temperature corrections
- measuring horizontal angles and zenith angles — direct and reverse and
- total stations — recording horizontal and vertical distance in direct and reverse modes

When systematic errors cannot be eliminated by procedural changes, corrections must be applied to the measurements. These corrections are computed from the fixed relations between the systematic error and the conditions of the observations. A simple example would be the temperature correction applied to a taped measurement. All systematic errors must be eliminated prior to any adjustment of a survey. This generally is accomplished by using correct procedures and mathematically correcting the measured value.

Random Errors A random error (or accidental error) is an error that does not follow any fixed relation to the conditions or circumstances of the observation. For a single measurement, it is the error remaining in the measurement after all possible systematic errors are eliminated.

Random errors are produced by irregular, complex causes that are beyond the control of the observer. Their occurrence, magnitude, and algebraic sign cannot be predicted. Each is truly random.

Random errors obey the laws of chance and are analyzed by the laws of probability. A complete discussion of the mathematical laws of probability is beyond the scope of this manual. The reference list in Appendix B includes some excellent publications that cover probability.

Theoretically, a random error has an equal chance of being negative or positive. Thus, errors of this type tend to be compensating. However, since the magnitude is also a matter of chance, random errors, to some degree, remain in every measurement.

Some examples of random errors are:

- The centering of an optical tribrach over a survey point. The surveyor attempts to center the optical plummet exactly. In reality, the tribrach may be left or right. This random error is referred to as centering error and is present whenever an optical tribrach, the traditional plumb bob, or a prism pole is used.

- The reading of a level rod. The surveyor typically reads the rod to the nearest hundredth of a foot (0.002m). The cross hair seldom is at an exact hundredth and, thus, the surveyor must make a decision.
- The reading and pointing of a total station. Surveyors do their best to align and read the instrument. In reality, the instrument may be pointing slightly left or right.

Mathematical corrections cannot be applied to random errors. The Department uses the compass method and least squares for adjustment of traverses. Such adjustments only provide an estimate of the best approximation of the true coordinate values. After an adjustment has been completed, each individual value, such as the coordinates of a specific point or the elevation, are still in error by an unknown amount. In other words, a mathematical adjustment of observations does not “fix” a poorly executed traverse or a level loop that contains systematic errors, excessive random error or blunders.

4.2.2.2 Sources of Error

There are three general sources of errors: personal, instrument, and natural.

Personal Errors Personal errors are caused by the physical limitations and observing habits of an observer. They can be either systematic or random.

Personal systematic errors are caused by an observer's tendency to react the same way under the same conditions. For example, a person may measure slightly long on every measurement because they always stand in a certain position when taping. Each observer makes a personal systematic error of a small degree on each individual observation. Fortunately, such errors are minimized by proper procedures.

Personal random errors are caused by the physical limitations, sight and touch, for example, of an observer. As with systematic errors, such errors are always present, to some small degree, in every observation. Common sense and attention to proper procedures generally keep such errors to a minimum.

Instrument Errors Instrument errors are caused by imperfections in the design, construction, and adjustment of instruments and other equipment. Some of these imperfections are:

- eccentricity of the instrument
- calibration of electronic distance measuring instruments (EDM/EDMIs)
- plate levels
- level line of sight is out of adjustment

Most instrument errors are eliminated by using proper procedures, such as observing angles direct and reverse, balancing foresights and backsights, and repeating measurements. Instrument errors that are not eliminated by procedures must be minimized by maintaining a regular program of periodically checking and adjusting (or calibrating) instruments and other equipment.

Natural Errors Natural errors result from natural physical conditions such as atmospheric pressure, temperature, humidity, gravity, wind, and atmospheric refraction.

Natural errors are removed from a measurement by determining corresponding corrections from known relationships between an error and the related natural phenomena. The ppm correction for temperature and pressure is a familiar example. The natural effect of curvature and refraction can be eliminated in leveling by balancing foresights and backsights, or in a traverse, by using reciprocal zenith angles.

4.2.2.3 How to Handle Errors

All surveying measurements are subject to systematic and random errors. Systematic errors need to be eliminated using proper procedure, or corrections must be applied to the measured values. It is preferable to eliminate systematic errors by using correct procedures. Examples of proper procedures are:

- adjust tribrach plummet
- traverses — measure all angles using both a direct and a reverse reading
- correct ppm settings on EDMs and
- level circuits — keep backsight and foresight distances approximately equal

After systematic errors are eliminated, random errors need to be addressed. Random errors occur during the course of any survey. They cannot be eliminated; nor can corrections be applied.

The compass adjustment and least squares adjustment is based on the theory that no systematic errors are present, and random errors tend to accumulate based on distances from the point of beginning. Traverse adjustments are not a valid method for distributing systematic errors or blunders.

A level circuit may not close exactly. Is the misclosure due to a systematic error, random error, blunder, or a combination of the three? The misclosure cannot be adjusted if it is due to a systematic error or blunder.

4.3 MAXIMUM ALLOWABLE CLOSURES

As mentioned earlier in this chapter, survey crews must eliminate systematic errors by calculations or correct procedures. Do not correct systematic errors by distribution of the error (horizontal or vertical). Blunders (mistakes) must be eliminated, and random errors must be held to a minimum.

The maximum allowable closures in the following sections can easily be attained using standard survey equipment provided to the Department's survey crews and equipment typically used by consultants. Closures (vertical and horizontal) should be substantially less than the maximum closures given. These maximum allowable closures have been adjusted to meet MDT design and construction needs.

4.3.1 LEVELS

4.3.1.1 Differential Levels

During the preliminary surveys, differential levels are used to establish elevations of control marks and photo control marks throughout the length of a project. They are generally the first levels on a project and are of the highest accuracy. The vertical closure associated with these levels may be due to systematic errors, random errors or mistakes. Since systematic errors and mistakes must be eliminated, the vertical closure should be due only to random errors. Random errors in levels are functions of distances or the number of turning points.

The maximum permissible closure for differential levels associated with project control marks is:

$$3 \times 0.005\sqrt{\text{Number of Foresights}} \qquad \text{(Feet)}$$

or

$$3 \times 0.0015\sqrt{\text{Number of Foresights}} \qquad \text{(Meters)}$$

For example, if your differential levels had 12 foresights, the maximum closure based on the above equation would be 0.052 ft (0.016m). This is then rounded to the nearest 0.01 ft or the nearest 0.001m, which is equal to 0.05 ft (0.016m). If the vertical closure is greater than 0.05 ft (0.016m), the level loop probably contains excessive systematic errors, excessive random errors and/or mistakes. The levels must be rerun.

If the error of closure is less than or equal to the maximum allowable closure, this error is then distributed. Adjust the elevations of all control marks and aerial photography control marks within the loop. Turning points do not need to be adjusted.

Single-wire levels using an automatic level may be adjusted based on turning points, but the sign of the adjustment must be taken into account. The closing error is prorated:

Vertical adjustment = (Error x No. of foresights to the mark) / Total No. of foresights

Single-wire level runs using a digital level may be adjusted as above, but the preferred method is to use a least squares adjustment such as Star*Net™ which is available from Star Plus Software, Inc. Star*Net allows adjustments to be preformed based on either turning points or distances.

4.3.1.2 Additional Vertical Control (Bench Marks)

At the completion of the control survey, all control marks and photo control marks will have three dimensional (3D) coordinates. During the construction phase of a project, additional vertical control (bench marks) may be required. These additional bench marks will consist of a 5/8-inch x 30-inch rebar (16mm x 762mm) with a Morasse® P43 (available from Berntsen Survey Markers) or equivalent driven over the rebar. Included in Chapter 10 is a picture of a Morasse® cap. When ordered from Berntsen the P43 should be pre-marked “MDT BM”. The BM number and date shall be stamped on the cap. The maximum vertical closures should be less than the permissible closure indicated above.

4.3.1.3 Other Levels

This category of levels consists of:

- differential levels from control points to aerial photography control points
- differential levels from known control points to new control points and
- trigonometric levels to aerial photography control points from known control

The vertical closures associated with this type of levels should meet the maximum permissible closure given above. Use of good leveling procedures should result in meeting this requirement. However, it is recognized that in certain situations, this maximum closure will not be met. In these situations, time spent rerunning loops is not warranted. Therefore, the absolute maximum vertical closures are:

- 0.10 ft (0.03m) for differential or trigonometric levels to aerial photography control points from existing control points.
- 0.05 ft (0.015m) for differential levels from known control points to new secondary control points or new secondary bench marks.

4.3.2 CONVENTIONAL CONTROL TRAVERSES

The control traverse associated with conventional control surveys serves as the primary horizontal control throughout the length of a project. It typically is the first traverse conducted and should be of the highest accuracy. As with vertical control, systematic errors and mistakes cannot be corrected by adjustment of a traverse. The errors must be eliminated before adjustment. The errors in a traverse must be the result of random errors only. The compass adjustment or least squares may be used to distribute random errors. If the compass adjustment is used, the angular error will be adjusted prior to the compass adjustment. Final adjusted coordinates should be compatible with the basis of bearing. If not, the traverse should be rotated to the basis of bearing.

Angular Closure. The maximum allowable angular closure in a control traverse is:

$$7 \text{ seconds x } \sqrt{\text{No. of Traverse Points}}$$

This product is rounded up to the nearest whole second. For example, if your control traverse had 15 traverse points, the maximum allowable angular closure would be 27.1 seconds. This is then rounded up to 28 seconds.

Closure After Angular Adjustment. The minimum traverse closure, after satisfying the angular condition, is 1:45,000, but the linear closure (closing distance) in no case can exceed 0.50 ft (0.150m).

4.3.3 SECONDARY TRAVERSES

This type of traverse is associated with ties to property controlling corners, property corners, aerial photography control points, establishment of additional control marks, and or traverses for topographical surveys. Secondary traverses will start at a known control point and will always close on a different known control point. The closing angle will always be measured. The compass adjustment or least squares may be used to distribute random errors. Elevations are not required if the traverse is used to obtain coordinates of property controlling and/or property corners.

Angular Closure. The maximum allowable angular closure associated with a secondary traverse is:

$$10 \text{ seconds} \times \sqrt{\text{No. of Traverse Points}}$$

This product is rounded up to the nearest whole second. For example, if the secondary traverse had five setups, the maximum allowable angular closure is 22.4 seconds. This is then rounded up to 23 seconds.

Closure After Angular Adjustment. The maximum linear closure, after satisfying the angular condition, is:

$$0.10 \times \sqrt{\text{Total Distance} \div 1000} \qquad \text{(Feet)}$$

or

$$0.03 \times \sqrt{\text{Total Distance} \div 304.8} \qquad \text{(Meters)}$$

In no case will the linear closure be greater than 0.60 ft (0.200m).

4.3.4 <u>RADIAL SURVEYS (SIDE SHOTS)</u>

Radial surveys may be used in lieu of a traverse to tie property controlling corners, property corners, aerial photo control points to the control traverse, or to establish additional control marks.

The maximum allowable radial difference between the horizontal coordinates of a monument must be less than or equal to 0.25 ft (0.08m). This radial difference is the distance (based on a horizontal inverse) between the two sets of coordinates of the monument tied. These coordinates usually are obtained from two control points, from a point on line (POL), or from one control point using different backsights.

If the radial method is used to establish additional control marks, then the maximum allowable radial difference in horizontal coordinates must be less than or equal to 0.10

ft (0.03m). The elevations of additional control marks will be determined using differential levels.

4.3.5 CELESTIAL OBSERVATIONS

A celestial observation is the preferred method to obtain a basis of bearing (bearing source) for non state plane coordinate projects. Generally, it is easier to schedule a solar observation as opposed to a Polaris (or other stars) observation. Either type of observation is acceptable.

Several sources of systematic errors can contribute to the accuracy of a solar observation. These may include determination of latitude, longitude, or time. A hand held GPS receiver will generally provide a NAD83 latitude and longitude to a sufficient accuracy, and time should be accurate to ±1 second. Sources of random errors are also present such as sighting error and the timing of the observations.

Maximum Differences After the completion of a solar observation, the individual direct azimuths to the sun should be determined. Any individual azimuth (outlier) greater than 20 seconds from the mean of all direct azimuths should be rejected. Determine each reverse azimuth to the sun. Any individual azimuth greater than 20 seconds from the mean of all reverse azimuths should be rejected. The final mean azimuth to the sun will be determined using an equal number of direct and reverse observations.

The mean azimuth as determined by the direct observations may indicate there is a significant difference (±40 seconds) when compared to the mean azimuth derived from the reverse observations. This difference is due to instrumental errors and will be eliminated in the final mean azimuth.

4.3.6 GPS SURVEYS

Refer to Chapter 8.

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Chapter 5
Conventional Control Surveys

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Chapter 5
Conventional Control Surveys

This chapter discusses conventional control surveys, requirements, and procedures. Conventional control surveys are accomplished using conventional methods and instruments such as total stations, levels, and data collectors. Control surveys are required for most Department projects. The control survey may be accomplished using conventional methods or Global Positioning System (GPS). Projects that may not require a control survey include overlays, bridge rehabilitation, guardrail installation, traffic, and safety projects and possibly others. However, if there is right-of-way involvement or the data collector will be used a control survey will be required.

Examples of other conventional preliminary surveys are:

- photogrammetric
- cadastral
- hydraulic
- topographic
- secondary traverses and radial ties

The main purpose of preliminary surveys is to gather all information that is required by Photogrammetry, Road Design, Bridge, Hydraulics, Utilities, Traffic, and Right of Way. This chapter discusses the procedures to be followed for conventional control surveys. Other chapters in this Manual cover the other type of preliminary surveys listed above. Appendix A contains sample field notes for various types of surveys discussed in the chapter. Even though survey crews typically use data collectors, the sample field notes may still be beneficial

5.1 PRELIMINARY FIELD REVIEW REPORT AND OTHER REPORTS

A Preliminary Field Review Report (PFR) is developed for each project. This report addresses such items as the project location and limits, existing physical characteristics of the bridge or the roadway, traffic characteristics, intent of the project (i.e. widening and slope flattening, total reconstruction), and major design features.

Included in this report is the type of survey required. It is important that the survey request be complete and appropriate

If the PFR indicates hydraulics involvement, the Location Hydraulic Study Report (LHSR) should be used to obtain the appropriate hydraulic survey data. Hydraulic surveys are discussed in Chapter 11

Based on the PFR, the lead bureau will prepare and distribute the Survey Request Form. The purpose of this form is to clarify and request specific survey information that may not be included in the PFR. This form includes such items as the use of aerial mapping, the coordinate system (state plane or a local coordinate system), vertical datum, hydraulic considerations, right-of-way involvement, and limits of the topographical survey. The PFR, the Survey Request, and the LHSR are available through the Department's Document Management System (DMS). In the event that any of these documents contain conflicting information, contact the designated OPX2 Project Manager to resolve the conflicts.

If either the PFR or the Survey Request indicates right-of-way involvement, right-of-way will prepare and distribute a survey request associated with the required cadastral survey. This request typically indicates the limits of the existing right-of-way to be retraced and monumented as well as required property controlling corners

5.2 CONTROL SURVEYS

A control survey is required for most types of MDT projects. Examples of projects that require a control survey are:

- photogrammetric projects
- new construction projects
- reconstruction projects
- bridge replacement projects
- some safety and traffic projects
- projects that utilize a data collector

The purpose of any control survey is to establish a series of semi-permanent, recoverable horizontal/vertical or vertical control marks throughout the limits of a project. Topographic features, controlling property corners, property corners, aerial photography control points (wing and center targets), right-of-way references, right-of-way monuments, apparent centerline, and other features are tied to the control survey. The control survey is also used to stake the final project.

Preliminary surveys generally follow the sequence covered below.

5.2.1 RESEARCH

Research is not unique to the control survey. Vital information can be obtained by searching available records. The research associated with the establishment of a new control survey may include, but is not limited to:

- as built plans
- United States Geological (USGS) quadrangle maps
- previous projects in or adjacent to the new project
- locations and descriptions of existing National Geodetic Survey (NGS) and USGS bench marks
- existing aerial photography
- existing horizontal control
- the PFR and other reports such as the LHSR
- investigate land ownerships for right to entry

5.2.2 RECONNAISSANCE

The survey crew should seek as much assistance as is required to get the control survey off on the right track. If there are questions, the District survey crew should contact the OPX2 Project Manager (PM), the Engineering Project Manager (EPM), Photogrammetry and Survey Section, or the District land surveyor. Consultants having questions should contact the appropriate Consultant Design Engineering Manager.

During reconnaissance, the survey crew should locate all relevant existing horizontal and/or vertical control. New projects may be within or adjacent to previous projects. In these cases, some or all of the existing control marks may be used for the new control survey.

Some questions to be asked are:

- How was the existing control established?
- Is the existing control in the best locations?
- Will it suit all the survey needs for this project through construction?
- Will a new control survey be required?
- Can some existing control marks be used and additional marks set?
- Where is the new project centerline probably going to be located?
- Where might the new construction limits be?

5.2.3 CONTROL POINTS (MARKS)

The District or PM will determine whether a new control survey is necessary or whether additional control points can be added to an existing control survey. Consultants, in conjunction with Consultant Design, will determine if new control is necessary or if additional control points can be added to an existing control survey.

This decision should take into account:

- accessibility of existing control marks
- estimated construction limits
- maximum and minimum distances allowed for control surveys
- safety of the field crew

- the overall probability that the existing monument will not be disturbed or destroyed during the construction of the project
- topography

5.2.3.1 Location of Control Points

The PFR should indicate the approximate location of the proposed centerline. This information will be helpful when selecting locations of control marks. If the PFR indicates the alignment will shift westerly of the existing alignment, the control marks on the west side should be further from the Present Traveled Way (PTW) than those on the east.

The ideal scheme will have control monuments along both sides of the PTW and all additional surveys can be completed without establishing additional control at a later date. It is suggested that control marks be within 1000 ft (300m) of the next closest mark, either on the same or opposite sides of the PTW. Terrain may require closer spacing of the control marks. In addition, this spacing will facilitate the ties to topographic features, existing monuments, photo control points, and eventually the layout of the staked centerline and associated right-of-way monumentation.

Preferred Configuration The preferred configuration of a control survey is one in which control marks are placed on both sides of the PTW. Control marks on opposite sides of the PTW should be staggered and not across from each other. The ideal location of a control mark would be one that is outside the estimated construction limits, in an area that is safe, and provides for collection of all required information in the immediate vicinity. If permission to enter private lands has been granted, the control marks can be placed outside the existing right-of-way limits. Figure 5-1 shows a typical traverse numbering sequence for two adjacent traverses.

Alternative Configuration Conventional control points are generally located along each side of the proposed project as shown in Figure 5-1. There may be cases when it is necessary to locate the traverse completely on one side of the proposed project or a portion of the traverse on only one side. Examples would be limited sight distance due to terrain, property owners not wanting survey crews on their land, excessive vegetation or obstacles (river) on one side of the road as opposed to the other side. In these cases, the alternate configuration can be used. Refer to Figure 5-2.

The forward and backward traverse may have to occupy nearly the same location on the ground. When these points are going to be close together, there is a potential for error in identifying or occupying the wrong point. One single point eliminates this problem. Figure 5-2 shows the common point F1234. The cap is stamped F1234. The point is identified in the field notes and data collector files as F1234.

The survey crew should also include additional observations (cross-ties) such as distances, or angles, and/or distances and angles between nearby control marks. For example, the horizontal angle E1234-X1234-D1234 and the associated distances between these control marks. The compass adjustment is not suitable in this case, since the crossties cannot be incorporated into the adjustment. Therefore, the traverse will be adjusted using a least squares adjustment program (i.e. Star Net™). All newly established or existing control points must provide for visibility ahead and back.

5.2.3.2 Numbering of Control Points

The numbering of the control points is alphanumeric. The alphanumeric designation is based on a letter followed by the Uniform Project Number (UPN). For example, AXXXX, BXXXX, CXXXX, where XXXX is the UPN. If the UPN is a five digit number, the control marks will then be stamped AXXXXX. The letter designation is the next available letter.

The first control point set on a new project where the UPN is 4567 would be A4567. The next control point set would be B4567. A4567 and B4567 can be on the same side of the existing alignment or they can be on opposite sides. In addition, they do not need to be sequential. Assume on day one, control marks K4567, L4567 were set. On day two it was determined an additional control mark was required between these two marks. This new control mark can be designated and stamped with the next available control designation. It is very important that duplication of mark names does not occur. The last control mark using the above procedures would be Z4567. If additional control marks are necessary, they would be AA4567, AB4567, and AC4567.

Caution Do **not** use the letters I, O or Q. These letters can be easily confused with numbers. All alpha characters are upper case. Do not use lower case characters

This identification number will be used in all field notes, data collection files, and will be stamped on the control cap. In addition, the year the monument was set will also be stamped on the control cap. The minimum stamp size accepted by the Department is 3/16”.

All data collectors and associated software must be capable of alphanumeric point numbering capabilities. If not, the data collector and/or software must be updated prior to commencing any survey for the Department.

Exceptions The above numbering scheme does not apply to National Geodetic Survey (NGS) marks, United States Geological Survey (USGS) marks, or found control marks from a previous MDT projects. If an existing NGS, USGS, or previous control point is to be incorporated into the control survey, or it will be incorporated into project levels; the mark designation in all data collector files and field notes will be the name as designated by NGS, USGS, or the previous MDT project.

For example, NGS bench mark E444 will be included in the control survey or it will be incorporated into the project levels, the mark name will be E444. The mark will not be stamped with a control point designation. An additional example would be a USGS bench mark 10JEB. If this mark is to be included in a control survey or it will be used in conjunction with project levels, the mark designation will be 10JEB. Total alpha control mark names will remain alpha such as HELENA GPS. Found monuments, such as property controlling corners, property corners or right-of-way monuments shall not be used as control points.

If a new project is within or adjacent to an older project and control marks from the previous survey are found, do not rename/restamp existing control marks. An example would be, a previous reconstruction project built several years ago and now a new project such as a bridge replacement falls within the limits of the reconstruction project. Existing control marks 44A, 44B, and 44C are to be used for the new bridge project. The existing point designations will remain as stamped on the cap. All new control marks would be stamped AXXXX, BXXXX, etc. If an existing point is destroyed or cannot be located, do not reuse the number. Prior to setting any new control marks, the survey crew should obtain information (coordinate values and abstracts) associated with any existing control points in the vicinity of the current project.

Non UPN Projects At this time, the Department does not have a uniform numbering system for control mark designations for projects that are not assigned a UPN (Uniform Project Number). For these projects, the control mark numbering system will be determined at the District level or in conjunction with Consultant Design. However, all control marks will adhere to the other guidelines of this manual. An abstract will also be required.

5.2.3.3 Control Monuments

The standard control monument is a 5/8-inch x **30-inch** (16mm x 762mm) rebar with an approved 2 inch (51mm) aluminum cap. This cap is pre-stamped with the Department's name and a triangle. The aluminum cap will be stamped with the control point number and the year. A witness post with the Department's decal will be driven as close to the monument as practical. The location of the witness post in relationship to the monument will be included in the abstract. The witness post should not be so close that it will interfere with setting the instrument over the monument. All new control points should be flush with the ground.

In an urban area, the setting of witness posts or standard control monuments may not be practical. PK nails, MagNails®, epoxied metal markers, etc, may be more appropriate in this situation. These control marks need to be semi-permanent, recoverable, and easily identifiable. Even though these marks may not have a stamped designation, they will be assigned a designation according to the manual. This designation will be used in all abstracts, data collection files, field notes, etc.

5.2.3.4 Additional Points

Temporary Points-Data Collection Only After the coordinates of the control survey points have been determined, it may be necessary to add temporary marks for the collection of topographic features. If this additional mark is to be used for data collection only, it can be either a hub and tack or a large spike. A rebar or a rebar with an aluminum cap should not be used. The designation can be any suitable name except a name reserved for control marks such as FXXXX. The elevation can be obtained using differential levels, or trigonometric levels (total station or a total station in conjunction with a data collector). Elevations based on trigonometric levels should be used with caution.

Control Densification-Points Prior to construction, and after the initial control survey has been complete, additional control marks may be required to complete the preliminary surveys. If the new marks can or will be used during construction staking, it will consist of a standard control aluminum cap and will be stamped with the next available control mark designation (RXXXX). Point numbers should not be duplicated. The horizontal coordinates will be obtained by a closed traverse or a radial survey (double tie) from existing control marks. The elevation is obtained by differential levels. Refer to Chapter 4 for traverse closures, radial differences, and vertical closures.

Once a project is let for construction, a considerable amount of time may have occurred between the time of the original control survey and the actual staking of the project. Original control marks may have been destroyed or obliterated. Additional control marks may be necessary, or existing control marks will be destroyed by construction activities. Additional control marks can be established by a closed traverse or double ties. Mark designations will not be duplicated. Elevations will be determined by differential levels. Refer to Chapter 4 for traverse closures, radial differences, and vertical closures.

An existing control mark in danger of being destroyed during construction may be referenced and then reset. The elevation of the reset mark will be determined by differential levels. Refer to Figure 5-3 for approved methods and Appendix A for sample field notes associated with reset marks. A control point reset from references will be monumented with a 2-inch (51mm) aluminum cap. The cap will be stamped with the original designation plus RESET and the year. For example “BXXXX RESET 2005”.

Field notes or data collection files are required for all additional control marks set during construction and/or all reset marks. The final asbuilt plans should include the new and reset control marks.

5.2.4 ABSTRACT

The abstract for control points is started at the time the control point is set. Separate pages of the field book are to be reserved for the descriptions. The survey crew has the responsibility to see that the description of the monument, at the time it is set, is suitable.

5.2.4.1 Reference Posts

On system routes (primary and secondary) generally have reference posts (mile posts) markers along the PTW. These markers may be used to determine the reference post (RP) of the control mark. If the survey vehicle is equipped with an onboard Distance Measuring Instrument (DMI), reference post to the control marks will be measured to the nearest 0.001 mile. If a DMI is not available, reference posts will be measured, using the vehicles odometer, to the nearest 0.05 mile. Reference post locations associated with a metric project will be based on miles and not kilometers. The reference post location in the abstract should reflect the precision of the measurement. For example, using the DMI a control mark was set at RP 10.300. The RP in the abstract will be shown as 10.300 and not 10.3.

Off system routes may or may not have reference post markers. If reference post markers exist, they should be used to determine reference posts associated with the control marks. If reference post markers do not exist, the location of a control monument in relationship to reference posts can be determined using MDT’s publication *Montana County Maps*. The maps are also available at http://www.mdt.mt.gov/. See the links to County Maps, Urban Plats or City Maps as appropriate. Both formats designate roads using a number and the letter B or E. The B indicates the beginning of the route (RP 0.000) and E indicates the end of the route.

For some urban projects, it may be impossible to determine a reference post location. In this situation, the location of control marks will be described relative to street intersections.

5.2.4.2 Location and Description

Information of all control marks will include the location and description of the monument. The location must include the reference post, (if available), the distance and direction from the PTW, and the location of the witness post relative to the control point. The description of the monument must include the size and type, whether the monument was found or set, and how the monument is stamped. Additional location information should be included, if available, such as distance and direction from semi-permanent objects such as fences, fence corners, power poles, and bridge ends. A hand compass is useful for determination of approximate directions. Most distances can be paced. Refer to the sample notes given in Appendix A.

Final Abstract A final abstract is created from field notes and placed in the appropriate project folder using the Document Management System (DMS). Consultants will submit this file to Consultant Design. This file is to be an American Standard Code for Information Interchange (ASCII) text file and not a MS Word® or MS Excel® file. **Upper** case letters are to be used. See Form 5-1 as an example.

5.2.5 DIFFERENTIAL LEVELS

After the control marks have been established, differential levels or the traverse can be started. The order does not make any difference and they can be done simultaneously. The purpose of differential levels is to establish elevations of all monuments included in the control traverse. All marks must be turned through. Intermediate foresights to these points are not permitted.

Levels associated with the control marks are generally the first levels on a project. The elevations determined at this time will serve as the primary vertical control for all subsequent differential levels. A digital level should be used in conjunction with a suitable level rod such as the Zeiss™ LD23 , or the Leica™ GKNL4m or GKNL4F. Extendable level rods such as the Zeiss ™ TD25 should not be used. Prior to the commencing of the differential levels, the level should be adjusted. During the levels an attempt should be made to balance foresight and backsight distances. The maximum distance for foresights or backsights shall be less than 200 ft (60m).

Whenever possible, existing National Geodetic Survey (NGS) or United States Geological Survey (USGS) bench marks are to be utilized. The preferred vertical datum for all new projects is the North American Vertical Datum of 1988 (NAVD88). Elevations associated with USGS bench mark are relative to the National Geodetic Vertical Datum of 1929 (NGVD29) and may have to be converted to NAVD88. The vertical datum (NGVD29, NAVD88, or assumed) must be cited in the field notes and any relevant correspondences.

The location of existing NGS and/or USGS bench marks relative to the project will determine how the differential levels are established. Different conditions may exist:

- Bench marks are present throughout the project at regularly spaced intervals.
- Bench marks are not present along the project but are nearby.
- Bench marks are not present along the project and they are not nearby.
- There are one or two bench marks for the entire length of a reconstruction project.

5.2.5.1 Maximum Vertical Closure and Adjustment

The maximum allowable closures for differential levels and the guidelines regarding how to distribute vertical closures by a proportional adjustment were discussed in Chapter 4. This adjustment method is typically used in conjunction with an optical level. If a digital level is used, it is recommended that vertical closures be distributed by a least squares adjustment. Star*Net™ is a least squares adjustment software that is available from Star Plus Software, Inc. If a least squares adjustment is used, a minimally constrained adjustment (elevation of one mark is fixed) should be run to verify that the vertical closures are acceptable. If so, a fully constrained adjustment should then be run to obtain final elevations. The fully constrained adjustment must be based on realistic standard errors.

5.2.5.2 NGS And/Or USGS Bench Marks Are Present Through Out the Project

Differential levels should commence at a found, identifiable bench mark and be run to the next found, identifiable bench mark. All convenient control points should have differential levels run through them at this time. The elevations of the NGS and USGS bench marks will be, in most cases, considered fixed. Return circuits are not necessary if you check into an existing bench mark within the tolerances given in Chapter 4. If the closures exceed the tolerances given in Chapter Four, continue to the next bench mark. If closures still exceed acceptable tolerances, additional differential levels shall be required.

The elevations of any omitted control points are determined, at a later time, using the adjusted elevations of the previously observed control points. The elevations of the omitted points may also be determined by appending this information to the previous raw level data and then run a least squares adjustment. Differential levels to the omitted marks will always commence at a mark of known elevation and will close on a different known mark.

The elevations of the control marks may be used as bench marks and may replace the need for having separate bench marks at approximately every 1000 feet. The traditional bench mark (vertical only), if required, is generally established during construction.

5.2.5.3 NGS And/Or USGS Bench Marks Are Not Present

If a NGS or a USGS bench mark is within two miles of any point of the project, that bench mark will be used. The elevation is brought into the project using differential levels. The differential levels must return to the starting bench mark, and the vertical closure must be less than the maximum allowable closure given in Chapter 4. This newly established bench mark would then be used to establish all other elevations associated with the project. An error in the elevation of this NGS or USGS bench mark due to settling or frost heave will not be detectable, but the relative differences in elevations will be correct.

If a NGS or a USGS bench mark is not within two miles of the project, the elevation of one control survey mark on the project can be determined using the Online Positioning User Service (OPUS), or an assumed elevation. Elevations of the remaining control marks are then determined using differential levels. Prior to adopting an assumed elevation, the Engineering Project Manager (EPM) or the consultant must check with the OPX2 designated PM or the appropriate Consultant Design Engineering Manager to verify if an assumed vertical datum is acceptable. Projects within a designated floodplain will not utilize an assumed vertical datum.

Differential levels should consist of loops of approximately 3000ft (900m) in each direction. Elevations of all convenient control points should be determined at this time. The forward run should include the control points that will be in the control traverse. The return to the beginning bench mark generally consists of only turning points. If least squares will be used, it is advisable to include some previously observed control and photo control marks in the return. The elevations of any omitted control points on the forward or return can be determined at a later time using the adjusted elevations of the previously observed control marks.

5.2.5.4 Limited Number of Government Bench Marks

It is impossible to discuss all the various combinations that can exist. General guidelines are: use all that are present, and adjust closures as explained in Chapter Four.

5.2.6 CONTROL TRAVERSE(S)

The control traverse serves as the primary horizontal control throughout the length of a project. Prior to commencing the control traverse, the electronic distance measuring instruments (EDM/EDMI) built into the total station must be checked against a NGS calibrated baseline. Refer to Chapter 3.

The control traverse consists of all control marks, all photo control marks, and NGS/USGS bench marks (if they can be occupied) near the PTW. Photo control marks (wing targets) are typically tied later using secondary traverse(s) methods.

5.2.6.1 Common Definitions Associated with Traverses

Closed Traverse A Closed Traverse is a traverse that begins at a known point and closes back to the same point, or closes on a different known point. Closing angles are always measured.

Control Traverse A series of one or more closed traverses. The control traverse is performed to the highest degree of precision used on a project. The control traverse marks are used as the horizontal and vertical datum base for all aspects of a project.

HI The height of the instrument is the vertical distance measured from the top of the control monument to the center of the horizontal axis of the instrument. This point is marked by a punch mark or dot on the instrument.

HS The height of the sight is most often the height associated with a prism. It is the vertical distance measured from the top of the control monument to the center of the prism.

Open Traverse Is a traverse that begins at one point and ends at another point, where the horizonotal relationship between the beginning and ending points is not known. This type of traverse is not acceptable.

Position A position is the act of making direct and reverse horizontal observations. A position consists of two direct and two reverse observations. More than one position may be required, depending on the survey requirements. A position is the term used when horizontal angles are measured using the directional method. The horizontal circle remains stationary for a given position

Set of Angles A set of angles is a series of horizontal angles where each observation is accumulated on the horizontal circle of the instrument. Half the set is measured in the direct mode and half in the reverse mode. This term is used when measuring horizontal angles using the repeating method. The repetition method is ***not*** to be used on control traverses or secondary traverses.

Set of Zenith Angles A set of zenith angles consists of a direct and reverse pointing of the instrument.

5.2.6.2 Instruments and Number of Positions Required for Control Traverses

Total stations are classified by their stated Deutsche Industrie-Normen (DIN). All control traverses associated with Montana Department of Transportation (MDT) projects will require total stations having DIN value no greater than three seconds. All horizontal angles will be measured to the right regardless of the size of the angle.

At each traverse point, four positions are required. The maximum difference in the horizontal angle measured direct and reverse for each position must be equal to or less than eight seconds, and each of the four position means must be within five seconds of the mean of the four positions. If either of these differences is exceeded, that position should be rejected (tossed). Only that position that was rejected needs to be returned. If a position was rejected, the new mean of the four good positions is computed. Refer to sample notes in Appendix A.

5.2.6.3 Basis of Bearing and Origin of Coordinates

All surveys must have a basis of bearing (bearing source). This bearing can be obtained from a celestial observation (solar observation, Polaris, or other stars), previous MDT projects (as built), or previous surveys (public or private). The land surveyor in responsible charge will make the final decision regarding the basis of bearing, unless the basis of bearing is designated either in the PFR or in the Survey Request.

Solar Observation A solar observation is the preferred method to obtain a basis of bearing. Appendix A shows the notes for one set of observations. It is suggested that a complete observation consists of two sets (each set consists of 4 direct and 4 reverse observations of the sun or selected star). The solar observation should be taken near the middle of the project.

The hour angle method will be used exclusively. Observations using Polaris or other stars are also acceptable. Solar observations should not be taken between 10:00AM and 2:00PM standard time. The reason being a diagonal eyepiece maybe required and accurate time becomes more critical. The survey crew should have available and refer to *Celestial Observation Handbook and Ephemeris* for the current year. This booklet is available from Sokkia™ or may be obtained from the Photogrammetry Survey Section. Items recorded in the field notes associated with a solar observation must include:

- Latitude and Longitude (NAD83)-obtained from scaling from a United States Geological (USGS) quadrangle map or hand held GPS receiver
- Date of observation
- Edge of sun observed (trailing-left edge or the leading-right edge)
- Greenwich Hour Angle-obtained from handbook
- Declination-obtained from handbook
- Semi-diameter-obtained from handbook
- UT1 = UTC + DUT at each pointing-obtained by short wave radio or by calling 1-303-499-7111

The determination of the direction between the instrument point and the backsight can be determined using the programs included in the handbook or the Tripod Data System (TDS) Ranger™. The results should be analyzed and outliers rejected (tossed). See Chapter 4 for maximum acceptable tolerances. The results of the celestial observation will be held in any subsequent horizontal adjustments (compass or least squares). If the compass adjustment is used, a rotation of the adjusted traverse to the basis of bearing (celestial observation) may be required.

Solar observations associated with Department projects will assume that geodetic latitudes and longitudes equals astronomic and the final azimuth from the celestial observation is a geodetic azimuth.

Origin of Coordinates Projects not relative to state plane coordinates require the coordinates of one control mark to be assigned assumed horizontal coordinates. The exception would be a new project that will utilize existing control from a previous project. The assumed coordinates should be large enough that negative coordinates do not occur. Projects must have only one basis of bearing and one coordinate system. All sections working on this project will use the same orientation and coordinate system developed by the Department or the consultant. The origin of bearings and coordinates should be stated in the field notes and included in appropriate correspondence

5.2.6.4 Horizontal Angular Measurements

Four positions are required. All angles will be measured to the right from the backsight. All backsights and foresights will be a prism secured in a tribrach fastened to a tripod. The extension of a prism using a pole is not permitted. The optical plummets are to be adjusted prior to commencing the control traverse. This should be done at the time of the baseline measurement.

Prior to moving the instrument, the surveyor should verify the correct number of positions are turned, the angles are meaned, and any positions that are rejected are returned. It is important that the instrument remain level throughout the series of measurements. This is the instrument operator's responsibility. If the total station is out of level, the instrument should be re-leveled and centered over the control point. Do ***not*** continue to measure angles.

When the required angular measurements are satisfied, the instrument is released from the tribrach (neither the tripod or the tribrach are moved) and replaced with a prism. The foresight prism is removed from the tribrach and the instrument is placed in that tribrach. The backsight, tripod, and prism are moved to the new foresight. The traverse is from tribrach to tribrach. The instrument and prisms are leapfrogged.

5.2.6.5 Distance Measurements

At each traverse point, the distance will be measured to the backsight and the foresight. The backsight distance must check the previous foresight distance within 0.02 ft (0.006m). It is important that the correct parts per million (ppm) and prism offset be set in the instrument. The prism should be mounted using the -30 mm offset and not the 0 mm offset. The EDM must have the same offset as the prism. The notes should show the temperature, pressure, and ppm set in the instrument. The information recorded depends on the type of instrument and how the reduction to horizontal will be made.

Methods Several methods can be used to determine horizontal distances between control marks. The method chosen depends upon the equipment used. Various methods are:

- record slope distances and the difference in elevation of the instrument and the prisms
- record slope distances and a set of zenith angles
- record measured horizontal distances. Curvature and refraction should be enabled

- a data collector which records heights, slope distances, and zenith angles (preferred method)

The first method requires the HI and HS to be recorded in all cases. The ground elevation of the control points must be known. Zenith angles are not required. Slope distances are measured to the backsight and the foresight.

The second method does not require HI's, HS's, or elevations of the control points. The slope distance and a set of zenith angles must be recorded to the backsight and the foresight. Do not reduce the slope distance to the horizontal and record only the horizontal distance. The slope distance and the zenith angles must be shown in the field notes. The sum of the direct and reverse zenith angles must show consistency. It is the note keeper's responsibility to verify consistency and inform the instrument operator if additional zenith angles are required.

The third method does not require recording HI's, HS's, slope distances, or zenith angles. The horizontal distances to the backsight and the foresight are recorded in the direct and reverse modes.

Maximum and Minimum Distances The maximum length of any leg in the control traverse is 2000 ft (600m). The minimum length should not be less than 200 ft (60m). In situations where the terrain limits sight distances to less than 200 ft (60m), the survey crew should take additional precautions to keep the centering error in the tribrachs to a minimum.

Maximum Traverse Lengths Traverse lengths (sum of all distances) should not exceed 4 miles (6.5km). In no case will the loop contain more than 20 points. Terrain limitations may require the length of the traverse to be shortened. All control marks, NGS and/or USGS bench marks, and aerial photo control marks near the PTW will be included in the control traverse(s). The exception to this is marks that cannot be occupied with the total station such as NGS bench marks set in vertical structures, NGS bench marks set in small protruding structures (headwalls), NGS bench marks set within railroad rights-of-way, and photo control marks located on the PTW. Typically the photo control points at some distance from the PTW will not be included in the control survey, but are tied using secondary control methods or approved radial surveys as outlined in Chapter 6.

5.2.6.6 Angular Closure

The angular closure that exists within a closed traverse must be due to random errors only. Systematic errors and blunders need to be eliminated prior to the adjustment of the angular error in a traverse.

The angular error is a function of the number of traverse points, horizontal distances to the backsights and foresights, atmospheric conditions, care in centering the tribrachs, precision of the instrument being used, stability of the instrument, and abilities of the observer. Angular error greater than the expected error cannot be adjusted. The angular error must be located in the field. See Chapter 4 for the maximum allowable angular error.

Angular error may be distributed using any of the following methods:

- Adjust all angles by equal amounts.
- Adjust the horizontal angles associated with the shorter distances more than the angles associated with longer distances.
- Adjust using the least squares method.

The first method is the most common and in most cases the preferred method, provided a least squares adjustment is not used. The second method requires some common sense and should not be used on a trial-and-error approach solely to reduce the linear error in a traverse.

5.2.6.7 Traverse Balance/Adjustment

Approved methods of traverse adjustment are

- compass
- least squares

The compass adjustment or least squares may be used to adjust traverses. If the project contains only one closed traverse, the adjusted coordinates will be essentially the same regardless of the adjustment method, provided the raw traverse closure is reasonable. If the project consists of two or more closed traverses, least squares is the preferred method to distribute random errors. However, the compass adjustment is also acceptable.

Compass Adjustment The compass adjustment requires the angular error to be distributed prior to adjustment of the traverse. All traverses, after the angular error has been distributed, must close within the maximum closures as stated in Chapter 4. If not, do not balance the traverse. The traverse probably contains systematic errors, excessive random errors, and/or blunders (mistakes). The errors or blunders must be located either in the field or office computations.

The compass adjustment requires a sequential adjustment of each traverse. The least squares adjustment performs a simultaneous adjustment of all traverses. The initial traverse adjustment should contain the two marks used to determine the basis of bearing and the control mark having assumed horizontal coordinates. After the successful compass adjustment of the initial traverse, a rotation will generally be required to conform to the basis of bearing. Each successive traverse is then adjusted holding the coordinates of two points from the previous traverse. Refer to Figure 5-4. If the traverses are being adjusted using the compass method, the coordinates of D1234 and J1234 are computed using the traverse information associated with traverse 3. These coordinates are now held to adjust traverse 4. These adjustments require the angles shown in Figure 5-4 at control points D1234 and J1234. Angular error associated with traverse 4 must be distributed prior to using the compass adjustment.

Least Squares Several commercial packages are available. Star*Net™ from Star Plus Software, Inc. is an example. It is suggested that a minimally constrained adjustment be preformed to verify traverse closures meet the closures (angular and linear) given in Chapter 4 and the raw data is free of mistakes. In a 2D minimally constrained adjustment, the North (Y) and East (X) coordinate of one point and an azimuth or bearing is constrained (fixed). In a minimally constrained 3D adjustment, the previous values are constrained plus the elevation of one mark. Either a 2D or 3D adjustment can be used. In most situations, it may be advisable to initially run a minimally constrained 2D adjustment first. The results of this adjustment should be reviewed and if acceptable, a 3D adjustment can be run.

The primary purpose of the 3D adjustment is to confirm elevations of the control marks as determined by differential levels. Any significant differences in elevations should be resolved. Once closures have been verified, elevations of control marks confirmed, and the results of the adjustment are acceptable a fully constrained adjustment (2D only) should be run. The fully constrained 3D adjustment is not required since final elevations of the control marks are based on the differential levels. In all adjustments, realistic standard errors must be used.

Least squares is the preferred method to adjust a series of closed traverses. All final 2D adjustments associated with control traverses should be adjusted simultaneously, and not sequentially. Redundant measurements should be incorporated into the traverse(s). See Figure 5-5.

Final Coordinates After the horizontal coordinates (2D) of the control survey have been determined using either the compass or least squares adjustment, the elevations based on the differential levels are appended to this coordinate file. Elevations derived from a 3D traverse are not included in the final coordinate listing. Horizontal coordinates will be shown to the nearest 0.001 ft (0.0001m). Elevations will be shown to the nearest 0.01 ft (0.001m).

5.3 DELIVERABLES

All traverse control data, including data collector files, computations, abstracts, field notes, and differential levels will be provided to the land surveyor in responsible charge. The land surveyor will finalize the control traverse. The land surveyor or designee will then upload all relevant files to the appropriate project folder using DMS. It is important that the finalization be done prior to distribution of the field data to other sections or bureaus.

If the control survey is in conjunction with a Consultant Design project, all the above data, with the exception of the field notes, will be provided to Consultant Design on a compact disk (CD). The original field notes will be submitted at the same time as all required digital files. All file names will conform to DMS file naming conventions. Consultant Design will forward this information to the Photogrammetry & Survey Section for finalization and uploading to DMS.

The following files are required and will be uploaded to the appropriate folder using DMS:

- all scanned right of entry forms
- all raw data collector files associated with the horizontal survey
- all raw data collector files associated with the vertical survey
- final ASCII text coordinate listing (format P N E Z) of all marks included in the control survey
- control abstract-ASCII text file
- all adjustment reports (horizontal and vertical)
- control diagram-consultant projects only
- read me file that explains the contents of all files
- In addition, a memo will be sent to the lead bureau indicating the control survey has been completed. All **original** field notes will be forwarded to the lead bureau or consultant design. Copies are not acceptable.

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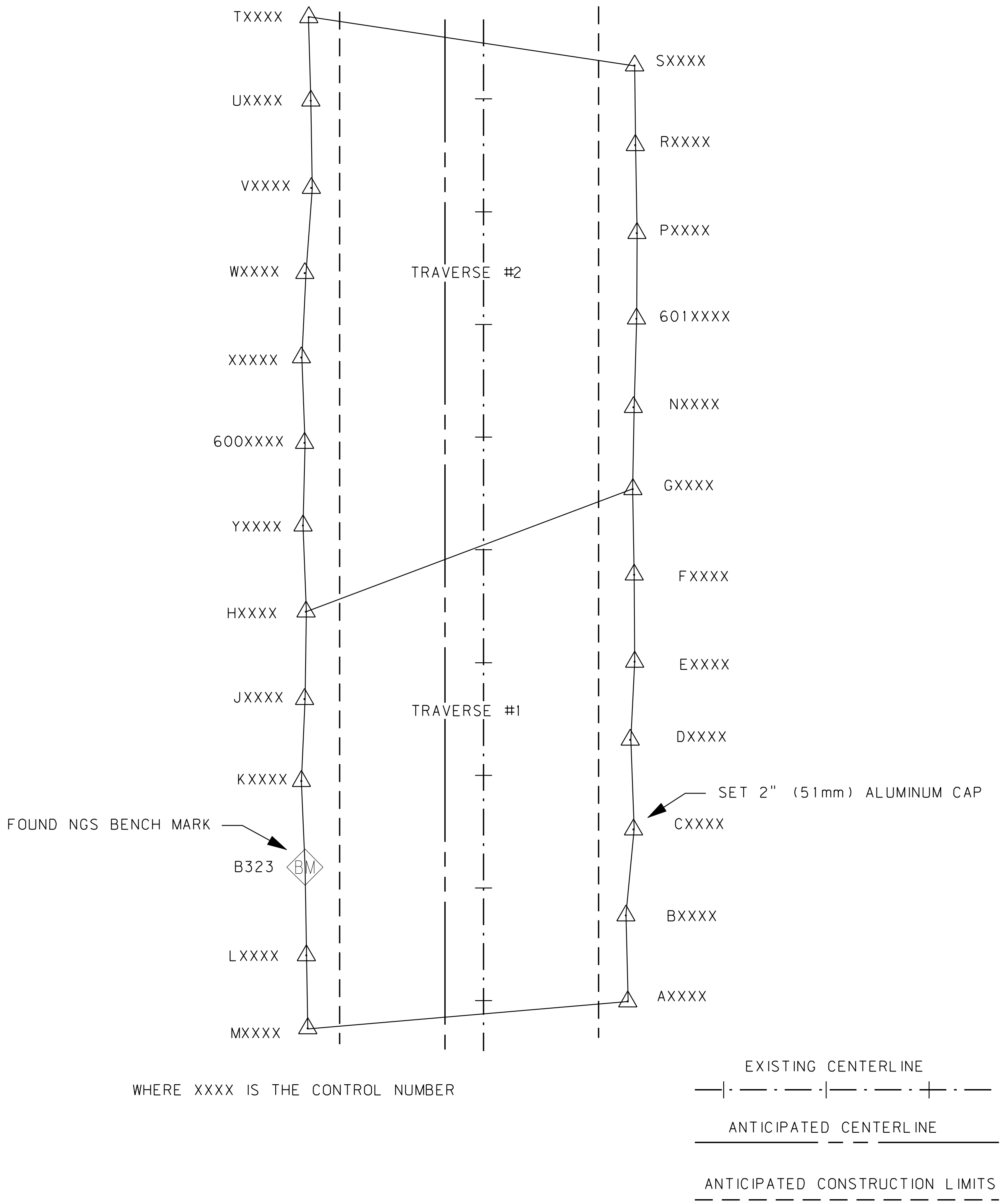

Figure 5-1
Typical Control Numbering Sequence

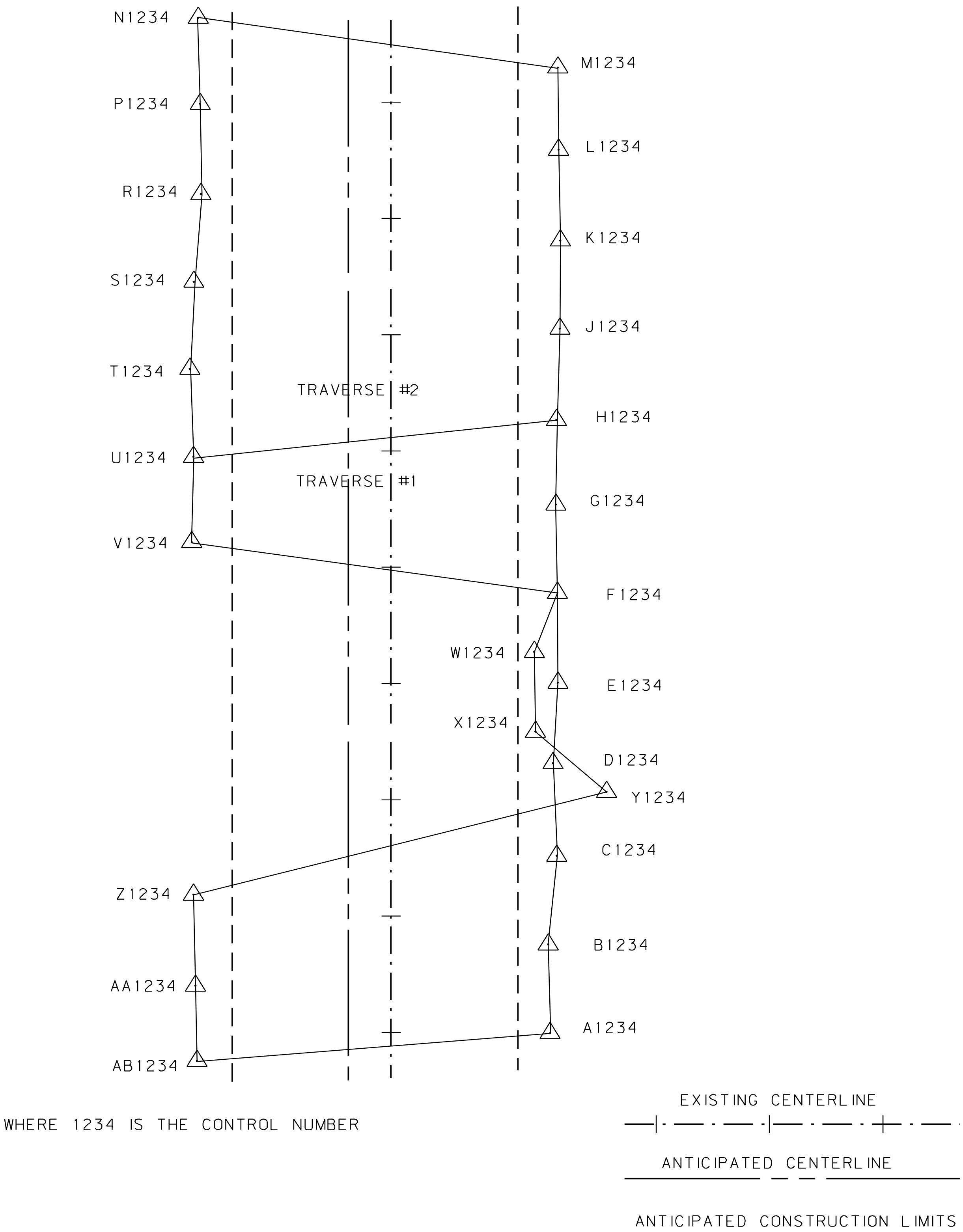

Figure 5-2
Alternate Control Numbering Sequence

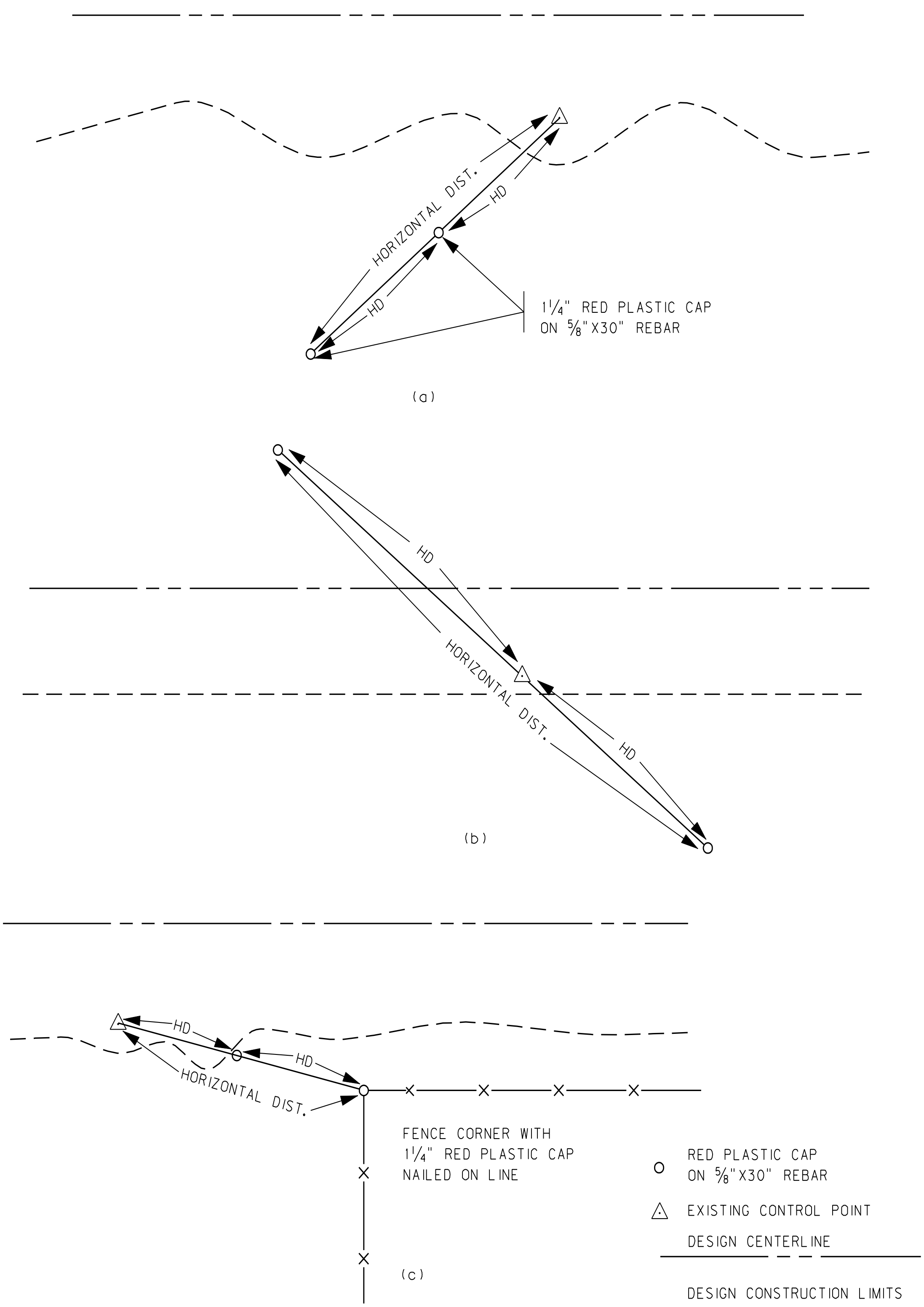

Figure 5-3
Control Point Referencing

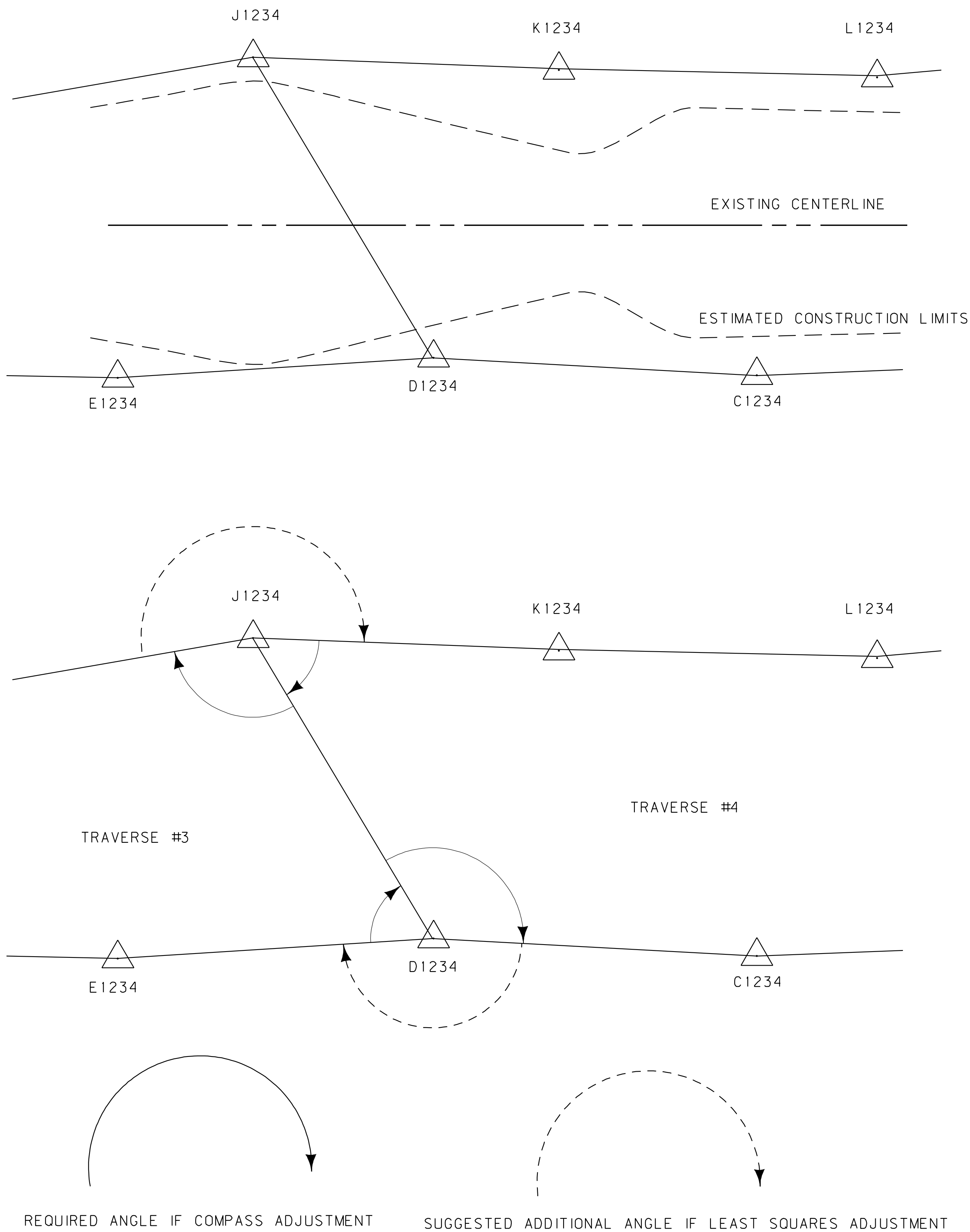

Figure 5-4
Closing Angles

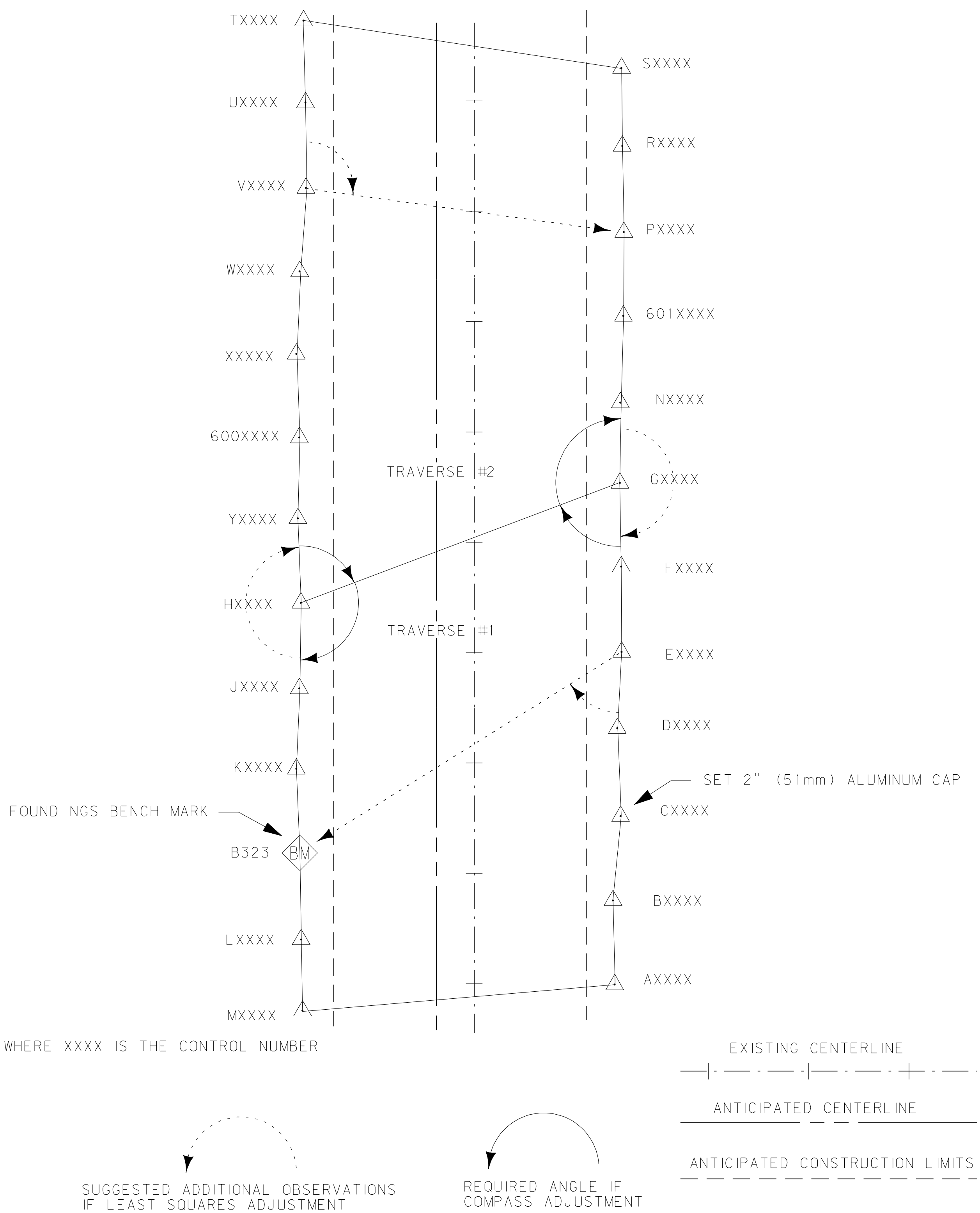

Figure 5-5
Redundant Measurements

CONTROL NUMBER: 9999
PROJECT ID: NH 9-99(99)999
LOCATION: SOMEWHERE IN MONTANA-EAST AND WEST

MARK DESCRIPTION

A5559 SET MDT ALUMINUM CONTROL CAP STAMPED "A5559 2005" ON A 5/8" REBAR, AT RP 399.123, 65' SOUTH OF PTW OF HIGHWAY 2, 5' SOUTH OF A FENCE, AND 5' SOUTH OF A WITNESS POST.

B5559 SET MDT ALUMINUM CONTROL CAP STAMPED "B5559 2005" ON A 5/8" REBAR, AT RP 399.450, 80' NORTH OF PTW OF HIGHWAY 2, 120' EAST OF A FARM ROAD, 3' NORTHWESTERLY OF A POWER POLE, AND 5' SOUTH OF A WITNESS POST.

T335 FOUND NGS BENCH MARK IN A CONCRETE POST STAMPED "T335 1938", AT RP 399.853, 55' NORTH OF PTW OF HIGHWAY 2, 5' NORTH OF A FENCE, AND 3' WEST OF A NGS WITNESS POST, SEE NGS DATA SHEET FOR ADDITIONAL INFORMATION.

C5559 SET MDT ALUMINUM CONTROL CAP STAMPED "C5559 2005" ON A 5/8" REBAR, AT RP 400.144, 80' SOUTH OF PTW OF HIGHWAY 2, 2'NORTH OF A FENCE, AND 5' SOUTH OF A WITNESS POST.

400A FOUND MDT ALUMINUM CONTROL CAP STAMPED "400A 1997", AT RP 400.329, SET IN CONJUNCTION WITH CN 2235, 82' NORTH OF PTW OF HIGHWAY 2, 3' WEST OF POWER POLE, AND 2' NORTH OF A FENCE, AND SET NEW WITNESS POST 3' SOUTH.

603 FOUND MDT ALUMINUM CONTROL CAP STAMPED "603 1997", AT RP 400.674, SET IN CONJUNCTION WITH CN 2235, 50' NORTH OF THE PTW OF HIGHWAY 2, 3' NORTH OF A FENCE, AND SET A NEW WITNESS POST 3' NORTH.

Form 5-1
Sample Control Abstract

Chapter 6
Secondary Traverses, Radial Surveys and Cadastral Surveys

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Chapter 6

Secondary Traverses, Radial Surveys and Cadastral Surveys

Conventional secondary traverses, radial surveys, or a combination of the these two methods may be used to determine coordinates of additional survey points and monuments that are required to complete several types of preliminary surveys such as:

- photogrammetric control points (targets)
- cadastral
- control densification
- hydraulic
- topographic

This chapter discusses secondary traverses, radial surveys (side shots), control densification, and cadastral (land) surveys. If the control survey marks were established at strategic locations during the control survey, the survey crew can now occupy these control marks and using radial survey methods complete the required surveys in the immediate area of the Present Traveled Way (PTW). As surveys extend farther from the PTW the coordinates of survey points and monuments may be required and radial survey methods cannot be used exclusively. Examples would be photogrammetric control points (aerial targets-wing), cadastral surveys, hydraulic surveys, and topographic surveys. These surveys may require the use of secondary traverses in conjunction with radial surveys.

6.1 SECONDARY TRAVERSE

A secondary traverse may be used to determine coordinates of additional points that are required in order to complete preliminary surveys. Total stations and data collectors should be used, but total stations and field notes are acceptable. Figure 6-1 shows an example of a secondary traverse to photogrammetric control point 603XXXX.

6.1.1 INSTRUMENTS AND NUMBER OF POSITIONS REQUIRED

Secondary traverses require total stations having a Deutsche Industrie-Normen (DIN) no greater than five seconds. Instruments having a DIN of not greater than three seconds are recommended. At each traverse point, two positions are required. A single position consists of two direct and two reverse observations. The maximum difference in the horizontal angle measured direct and reverse for each position must be equal to or less than twenty seconds and each position mean must be within ten seconds of the mean of both positions. If either of these differences are exceeded, the horizontal angle is returned until these angular conditions are satisfied.

6.1.2 HORIZONTAL ANGULAR MEASUREMENTS

All horizontal angles will be measured to the right (clockwise) from the backsight. All backsights and foresights will be to a prism secured to a tribrach and tripod. Prior to moving the instrument, the surveyor shall verify the correct number of positions was turned and all angular conditions given above are satisfied.

6.1.3 DISTANCE MEASUREMENTS

At each traverse point the distance will be measured to the backsight and the foresight. This includes the initial backsight and the final foresight. The initial backsight distance and the final foresight distance should agree within 0.10 ft (0.030m) of the inversed distance. All other backsight distances must check the previous foresight distance within 0.03 ft (0.010m). In most cases elevations of the secondary traverse points are not required; therefore, heights of instruments (HI) and height of sights (HS) are not necessary. The HI and HS must be recorded when elevations are required.

If data collectors are used, all measured zenith angles and slope distance will be recorded. However, when field notes are used then either slope distances and zenith angles or horizontal distances shall be recorded. Do not measure slope distance and manually convert and record the horizontal distance. The recording of horizontal distances requires curvature and refraction be enabled in the total station. The horizontal distances will be recorded both direct and reverse. These two horizontal distances must agree within 0.03 ft (0.010m).

6.1.4 NUMBERING AND POINT DESIGNATIONS

Secondary traverse points will be assigned a number from 1 to 199. This number will be recorded in the data collector and/or in the field notes. Alphanumeric point numbering will not be used. Alphanumeric numbering is reserved for control mark designations. The point number in field notes and/or data collector files will be compatible with the number assigned to that point.

Typical secondary traverse points are large nails, a railroad spike with a punch mark, a hub and tack, or other suitable material. A rebar and aluminum cap is not required. The description of the traverse points should be recorded in field notes and/or the data collector. This is particularly important if the secondary traverse will be used in conjunction with the cadastral survey. It is suggested a stake or lath be placed next to the traverse point with the corresponding traverse number written on the stake.

The following point numbers are reserved:

- 1-199 secondary traverse points
- 200-299 property controlling corners
- 300-599 property corners, reference pins, and right-of-way monuments
- 600-799 photogrammetric control points
- 800-999 non specified but reserved

6.1.5 TRAVERSE ADJUSTMENT

Secondary traverses will always be closed and will include the closing angle. See Figure 6-1. The secondary traverse will commence at a known control mark with a backsight to another control mark and will close on a **different** control mark. The secondary traverse will not commence and end on the same control mark. The final observations recorded will be the closing angle and distance to the last observed control mark. In Figure 6-1, the angle at GXXX is the closing angle, and distance GXXXX to FXXXX is the closing distance. Total traverse lengths should be less than four miles (6400m).

The compass adjustment or least squares may be used to adjust secondary traverses. The compass adjustment requires the angular error to be distributed prior to the adjustment of the traverse. Refer to Chapter 4 for maximum angular error and linear closure.

If a least squares adjustment is used, a minimally constrained adjustment should be preformed to verify traverse closures (angular and linear) are acceptable. Once closures have been verified, a fully constrained adjustment should be run. Realistic standard errors must be used for this adjustment.

6.2 RADIAL SURVEYS

Radial surveys (side shots) serve the same purpose as secondary traverses. They are used to locate additional points needed to complete various preliminary surveys. The horizontal specifications in this chapter do not pertain to the establishment of a temporary point that will be used solely for collection of topographic features.

Radial surveys may be used to determine the required information associated with photogrammetric control marks, densification of the existing control, and existing monuments, including right-of-way reference pins, right-of-way monuments, property corners, and property controlling corners. Radial surveys may also be used in conjunction with a secondary traverse. Refer to Figure 6-2.

As mentioned in Chapter 5, the control traverse should include all control marks, all photogrammetric control marks, and all National Geodetic Survey/United States Geological Survey (NGS/USGS) bench marks (if they can be occupied) near the PTW. An exception would be photogrammetric control marks located on or close to the PTW. An example would be a photogrammetric control point that was placed at an existing bridge end. The occupation of this point during the control survey would create a dangerous situation for the instrument person. The radial survey method can be used in lieu of a traverse through this photogrammetric control point. Radial surveys should utilize data collectors and total stations. Total stations and field notes are acceptable.

6.2.1 INSTRUMENTS AND NUMBER OF POSITIONS

Radial surveys require total stations having a Deutsche Industrie-Normen (DIN) no greater than five seconds. Instruments having a DIN of not greater than three seconds are recommended. At each occupied mark a minimum of one position is required. A single position consists of two direct and two reverse observations. The maximum difference in the horizontal angle measured direct and reverse must be equal to or less than ten seconds. If the angular difference is greater than ten seconds the horizontal angle is returned. For those surveys associated with the densification of the existing control survey it may be advisable to obtain two positions, instead of the minimum of one position. When the survey uses two positions then the maximum difference in the horizontal angle measured direct and reverse for each position must be equal to or less than twenty seconds and each position mean must be within ten seconds of the mean of both positions. If either of these differences are exceeded, the horizontal angle is returned until these angular conditions are satisfied.

6.2.2 HORIZONTAL ANGULAR MEASUREMENTS

All horizontal angles will be measured to the right (clockwise) from the backsight. All backsights will be to a prism secured to a tribrach and tripod. The foresight may be a prism pole, unless the foresight is associated with densification of the existing control survey. In this case a prism attached to a tribrach and tripod should be used. The surveyor should verify the angular conditions given above are satisfied.

6.2.3 DISTANCE MEASUREMENTS

Prior to measuring/recording information to the foresight, a distance must be measured to verify the backsight. This distance should agree within 0.10 ft (0.030m) of the inversed distance. In most cases, elevations of the foresight points are not required; therefore, heights of instruments (HI) and heights of sights (HS) are not necessary.

If data collectors are used, all zenith angles and slope distances will be recorded. However, if field notes are used in lieu of a data collector then slope distances and zenith angles are recorded or horizontal distances may be recorded. Recording of horizontal distances requires curvature and refraction be enabled in the total station. If horizontal distances are recorded in the field notes this horizontal distance will be recorded both direct and reverse. The two horizontal distances must agree within 0.03 ft (0.010m).

6.2.4 NUMBERING AND POINT DESIGNATIONS

Refer to 6.1.4 for point numbers. The description of the monument should be recorded in field notes and/or the data collector. To avoid errors in numbering of reference pins, right-of-way monuments, or property corners a stake or lath should be driven next to the monument with a number that coincides with the number used in the field notes and/or the data collector.

6.2.5 APPROVED RADIAL SURVEY METHODS

No single observation will be used to determine coordinates of any photogrammetric control point, property controlling corner, property corner, right-of-way reference pin, or right-of-way monument. All marks will be observed at least twice. Approved survey methods are:

- radial survey from two control points or two secondary traverse points
- radial survey from one control point and a random point
- radial survey from one control point, but different backsights

6.2.5.1 Radial Tie From Two Control Points or Two Secondary Traverse Points

This is the preferred method. This method can be used to establish the coordinates of monuments from either a secondary traverse or from the control survey. Refer to Figure 6-2. As indicated in this figure, photogrammetric control point 6035344 was tied from two different secondary traverse points (18 and 19).

Figure 6-2 also shows this method being used to obtain the coordinates of photogrammetric control point 6025344 from two different control points (G5344 and H5344). Photogrammetric control point 6025344 could have been included in the control survey. In this case it was not, since this would have required the survey crew and equipment to be placed on the highway.

Figure 6-5 is an additional example of this method. The control survey had been completed, but in order to finish additional preliminary surveys new control marks (W7121 and X7121) were required.

6.2.5.2 Radial Tie From One Control Point and a Random Point

This method may be used if the foresight point is not visible from two different control marks or from two different secondary traverse marks. Refer to Figure 6-3. This figure indicates this method being used to observe two different photogrammetric control marks. The first tie is made with the instrument at AD7989 and a backsight to AC7989. The required angles and distances are measured to photogrammetric control points 6037989 and 6047989 as well as to random point 57. Random point 57 may be set on the line between two control marks (AD7989 to AE7989) or set at a convenient location near AD7989 as shown in the figure.

The instrument is then moved to random point 57 and a backsight is taken on the furthest visible control point from random point 57. Using AE7989 as the backsight, the required angles and distances are then measured to photogrammetric control points 6037989 and 6047989.

6.2.5.3 Radial Tie From One Control Point and Different Backsights

The two previous methods are preferred, but radial ties using this method are acceptable. Refer to Figure 6-4. Secondary traverse point 41 was established using a secondary traverse. This figure shows a radial tie to photogrammetric control point 60310123 using different backsights (X10123 and Z10123). This figure also shows this method to tie photogrammetric control point 60110123 from control mark X10123. Once again, different backsights (Y10123 and W10123) were used.

6.2.6 RADIAL SURVEY COORDINATES

The three methods above provide the necessary information to determine two independent sets of coordinates for each radial foresight point. In general, the radial error between the horizontal coordinates must be equal to or less than 0.25 ft (0.076m). However, if the radial survey method is used to established new control marks, then the radial error between the horizontal coordinates must be equal to or less than 0.10 ft (0.030m). If these conditions are satisfied, the average (mean) horizontal coordinate may be used in any final coordinate listings.

In lieu of a mean coordinate, a least squares adjustment may also be used to determine the coordinates of the foresight points. It may be advisable to run a minimally constrained adjustment to verify all radial tolerances are satisfied. Once the radial tolerances are verified then run a fully constrained (2D) adjustment to obtain final coordinates of all foresight points. Realistic standard errors must be used for least squares adjustment.

6.3 ELEVATIONS

Traverse points and monuments located using secondary traverses or radial survey methods may not require elevations. Exceptions are additional control marks (densification) established after the completion of the control survey, photogrammetric control marks (targets), and random points necessary for topographic surveys.

6.3.1 DIFFERENTIAL LEVELS

The elevations of photogrammetric control marks located near the PTW and all new control marks established after the completion of the control survey will be determined by closed differential level procedures. The levels should commence at an existing control mark and close on a different control mark. Refer to Chapter 4 for maximum allowable closures.

Differential levels are the preferred method to determine elevations of all photogrammetric control points and should be used in most cases. Refer to Figure 6-6. The levels should not commence and close on the same control mark. The elevations of photogrammetric control marks will not be determined based on an intermediate foresight (side shot). Either an optical or a digital level may be used. Final elevations are based on a proportionate adjustment of the vertical error or least squares. See Chapter 4 for maximum allowable closures.

6.3.2 TRIGONOMETRIC LEVELS

The elevations of photogrammetric control marks that are not near the PTW or will not be used for staking may be determined using differential levels, trigonometric levels, or a combination of trigonometric levels and differential levels. The horizontal coordinates and elevations (3D coordinates) of these points are required for mapping.

The coordinates (3D) of photogrammetric control points may be determined using the secondary traverse method provided there will be no more than five instrument setups including the first and last setup. Refer to Figure 6-7. Either a data collector in conjunction with a total station or a total station and field notes may be used.

All heights of instruments (HI) and heights of sights (HS) must be measured and recorded in either field notes or the data collector. A suggestion is to measure and record these heights in both feet and metric units. Heights should be recorded to the nearest 0.01 ft and 0.002m, respectively. Curvature and refraction should be enabled in both the total station and the data collector. Reciprocal vertical information should be obtained.

A total station and data collector survey will record all observations including the height of the instrument and the prisms. A total station and field note survey requires the recording of slope distances and zenith angles **or** horizontal and vertical distances as displayed in the total station. In both cases the HI's and HS's must be recorded. If vertical distances as displayed in the total station are recorded, then the vertical distances must be recorded in both direct and reverse mode (face). The difference in these vertical distances must be equal to or less than 0.05 ft (0.015m). To achieve this tolerance, the instrument's zenith circle may need to be indexed (collimation error). Regardless of the method the effect of curvature and refraction must not be applied twice.

Curvature and refraction (c&r) corrections must be applied in certain situations. General formulas to compute c&r and elevations are given below:

$$c\&r = 0.0206M^2 \quad \text{Where M = distance in thousands of feet}$$

or

$$c\&r = 0.0675KM^2 \quad \text{Where KM = distance in kilometers}$$

and

$$\text{Elevation = Elevation at Instrument + HI + (VD + c\&r) - HS}$$

Attention must be given to the sign of the vertical distance in the above formula.

The horizontal coordinates may be determined using the compass adjustment and elevations may be determined by a proportionate adjustment of the vertical error. The coordinates may also be determined by 3D least squares adjustment. Closures and specification given earlier apply.

Elevations of points that will be occupied **only** for topographic surveys may be determined using trigonometric levels. Curvature and refraction should be enabled in the total station and data collector. All heights of instruments and prisms must be measured. Elevations based on trigonometric levels should be used with caution.

6.3.3 DIFFERENTIAL LEVELS IN CONJUNCTION WITH TRIGONOMETRIC LEVELS

This method consists of running differential levels from a control mark to the photogrammetric control point. A return is not required. Then using trigonometric levels the elevation of this same point is determined. The elevation obtained from the differential levels and the elevation obtained from the trigonometric levels must agree within 0.10 ft (0.030m). The elevation from the differential levels will be held.

6.3.4 COORDINATE VALUES

Horizontal coordinates obtained using either secondary traverses or radial ties shall be shown to the nearest 0.001 ft (0.0001m). All required elevations should be shown to the nearest 0.01 ft (0.001m).

6.3.5 SUMMARY

The following summarizes the horizontal and vertical requirements associated with secondary traverses and radial surveys.

I. Horizontal Requirements.
- a. Secondary Traverse.
  - i. Distance Requirements.
    1. Initial backsight and final foresight distance.
       - a. Equal to or less than 0.10 ft (0.030m) of the inversed distance.
    2. Difference between the remaining backsight and foresight distances.
       - a. Equal to or less than 0.03 ft (0.010m) when compared to each other.
  - ii. Horizontal Angle Requirements.
    1. Closing angle required.
    2. Two positions at all traverse points.
       - a. Difference between direct and reverse equal to or less than 20”.
       - b. Difference between mean of each position equal to or less than 10” of the mean of both positions
  - iii. Traverse Closure.
       - a. Maximum angular closure equal to or less than 10” x $\sqrt{No.ofSetups}$

b. Maximum linear closure after satisfying the angular condition 0.10' x $\sqrt{TotalDist \div 1000'}$ or 0.030m x $\sqrt{TotalDist \div 304.8m}$

iv. Coordinates Determined by Compass or Least Squares

b. Radial Surveys.

i. Distance Requirements.

1. Backsight distance equal to or less than 0.10 ft (0.030m) of the inversed distance.

ii. Horizontal Angle Requirements.

1. Minimum of one position.

a. Difference between direct and reverse equal to or less than 10".

iii. Radial Difference in Horizontal Coordinates.

1. Property corners equal to less than 0.25 ft (0.076m).
2. Control densification equal to or less than 0.10 ft (0.03m)

iv. Coordinates Determined by Mean Value or Least Squares.

II. Vertical Requirements.

a. Differential level tolerance less than or equal to 0.10 ft (0.030m).
b. Differential levels compared to trigonometric levels to the same point less than or equal to 0.10 ft (0.030m).

6.4 CONTROL DENSIFICATION SURVEYS

At the time of the original control survey (conventional or GPS), control marks should have been placed at all locations necessary to complete the preliminary surveys. Additional control marks should not be required. However, as the preliminary surveys are completed, additional control marks may be necessary. Refer to Figure 6-5. The new control marks will consist of a rebar (5/8-inch x **30-inch** (16mm x 762mm), an appropriately stamped 2 inch (51mm) aluminum cap, and a witness post. An abstract is required.

The horizontal coordinates of the new control marks can be established using secondary traverse procedures or radial survey methods. Final traverse horizontal coordinates are derived from either a compass adjustment or least squares. Final radial survey horizontal coordinates are based on either a mean coordinate value or least squares. Differential levels are required to determine elevations.

A land surveyor will finalize all control densification surveys. The land surveyor or designee will then upload all relevant files to the appropriate project folder in the DMS.

6.4.1 REVISED CONTROL DIAGRAM

Once the conventional control survey (Chapter 5) has been completed and the required deliverables have been placed in the appropriate project folder, the control diagram can be completed. The control diagram for Department projects are typically created by Road Design or Traffic. The control diagram for consultant projects are completed by the consultant. The original control diagram will not include the new control marks associated with the control densification surveys. Therefore, this control diagram should be updated to reflect all control marks associated with the project. The control diagram will **not** include the random points that were established in conjunction with topographic surveys.

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6.5 CADASTRAL (LAND) SURVEYS

All cadastral surveys will be completed under the direct supervision and responsible charge of a land surveyor licensed to practice in the State of Montana. Direct supervision is defined in the Administrative Rules of Montana (ARM) Title 24, Chapter 183, Sub-Chapter 3. The land surveyor will also be in responsible charge of the survey as defined in ARM Title 24, Chapter 183, Sub-Chapter 3, and Montana Code Annotated (MCA) Title 37 Chapter 67. Cadastral surveys will conform to all applicable state statutes and ARM. Cadastral surveys are required for Department projects that will require additional right-of-way. The Department's existing right-of-way may be in the form of an easement or may have been obtained in fee. Right-of-way as used in this Manual can refer to either an easement or fee. No distinction is made between the two.

6.5.1 GENERAL

Montana is one of the thirty public land states originally surveyed under the direction of the United States General Land Office (GLO). In 1946, the GLO was abolished, and the administration, including surveys, resurveys, and maintenance of all survey records was officially passed to the Bureau of Land Management (BLM) under the Cabinet Post of the Secretary of the Interior. Original and subsequent survey records are kept in the State Office of the BLM in Billings. By the time the rectangular survey system for the public land surveys reached Montana, rules outlining the survey methods, and the numbering and subdivision of the sections within each 36 square mile township had been developed.

The purpose of the public land survey was to provide a method for the government to sell, or otherwise dispose of, lands in the largely unsettled West. The surveys were designed so that a Patent Deed, which included reference to the survey on the ground, could describe land. This system formed the basis of all subsequent property surveys and conveyances of property in the public land states. Monuments set during the original public land survey control subsequent surveys and locations of described properties.

All projects that require additional right of way may require ties to property corners and property controlling corners. A property controlling corner is a public land survey corner or any property corner which does not lie on a property line of the property in question but which controls the location of one or more of the property corners of the property in question. Examples are section corners, quarter corners, closing corners, meander corners, witness corners, etc. that monument the rectangular survey system. Property corners monument individual tracts of land.

The primary purposes of cadastral surveys are to retrace and monument the record location of the existing right-of-way and to locate or establish the required property controlling corners. These surveys are an integral part of the land acquisition, disposal, use, and maintenance of our right-of-way records.

Cadastral surveys commence with a request from the Right-of-Way Bureau or District Right-of-Way Design and are not complete until the existing right-of-way is monumented, corner recordations and certificate of survey(s) are recorded. The Right-of-Way Bureau or District Right-of-Way Design will be provided copies of all recorded documents associated with the cadastral survey.

6.5.2 CADASTRAL REQUEST

The Preliminary Field Review (PFR) will indicate if additional right-of-way will be required. If additional right-of-way is required, Right-of-Way will then distribute their initial cadastral survey request. This request includes the preparation of a certificate of survey and corner recordations. The request also indicates required property controlling corners for the project. Generally, the nearest property controlling corner on each side of all section line crossings will be required. This request also indicates the limits of the retracement of the existing right-of-way. As a minimum, a retracement of the existing right-of-way is required at the beginning and end of the project. A retracement of the existing right-of-way for the entire limits of the project may be requested if there is a possibility of the utilization of the existing right-of-way at any location. Small projects such as a bridge replacement generally require a retracement of the existing right-of-way for the entire limits of the project. Right-of-Way should make the final determination as to which property controlling corners are required, and the extent of the retracement of the existing right-of-way

As a project proceeds through design, Right-of-Way may request additional cadastral surveys. These additional requests typically require additional retracement of the existing right-of-way in the areas were existing and new right-of-way intersect. The purpose of the additional request is to eliminate the need for a statement in the right-of-way plans indicating, “right-of-way equals existing.”

Some off system projects will be located on county roads. If the width of the road can be determined based on recorded documents this width will be monumented. If there is no information associated with the width of the county road, the centerline will be monumented.

The Right-of-Way Bureau or District Right-of-Way Design will not request a cadastral survey if Consultant Design administers the project. However, it may be in the best interests of the consultant and the Department if they consult with the Right-of-Way Bureau. The consultant is responsible for all cadastral surveys necessary for the development of right-of-way plans.

6.5.3 RESEARCH

Generally research is required prior to any field work associated with the cadastral survey. Most current highway projects will follow the general alignment of existing roads. Research may include but is not limited to the following:

- BLM field notes and plats
- railroad maps and plats
- Department records from previous projects (construction and right-of-way plans, original field notes, and deeds)
- aerial photography.
- other state and federal agencies documents (Forest Service, Bureau of Indian Affairs)
- county courthouse
- United States topographic maps

6.5.4 NUMBERING

As outlined in Section 6.1.4, property controlling corners will be assigned a point number from 200 to 299. Property corners will be assigned a point number from 300-599. Property corners include but are not limited to monuments set by land surveyors associated with property lines, reference pins, and right-of-way monuments. The assigned point number will be consistent in all field notes, data collector files, coordinate lists, and as shown on any corner recordations and certificates of survey.

6.5.5 CORNER TIE PROCEDURES

During the retracement survey of either the existing right-of-way or the dependent resurvey associated with the property controlling corners, all monuments (right-of-way, reference, property corners, and property controlling corners) will be located by either secondary traverse or radial survey methods. Refer to Figures 6-1 through 6-4. All relevant found monuments are to be shown on the certificate of survey.

6.5.6 MONUMENT DESCRIPTIONS

All monuments must be described fully. Descriptions in data collector files do not provide enough detail; therefore, descriptions must be included in field notes. The information in the field notes may include a rubbing, sketch of the monument, size, kind, and type of monument, and other identifying information. The field notes will indicate whether a monument was found or set. A digital photograph may be useful.

6.5.7 EXISTING DEPARTMENT RIGHT-OF-WAY MONUMENTS

Different combinations of monuments and locations of monuments have been used by the Department. One common situation is a reference pin and a concrete right-of-way monument at breaks in the right-of-way and at a various centerline control points such as point of curvature (PC) or point of tangency (PT). The reference pin is generally located 3 feet inside the right-of-way. The right-of-way staking notes or the as built plans may indicate this offset distance. It may be beneficial to confirm the approximate offset of the reference pin or the right-of-way monument from the roadway centerline (PTW).

6.5.8 COORDINATES

The procedure to compute and adjust a secondary traverse or a radial survey associated with the cadastral survey is identical to the methods previously discussed in this chapter. An American Standard Code for Information Interchange (ASCII) text coordinate file will be prepared that includes coordinates and descriptions of all found and set monuments associated with the cadastral survey.

6.5.9 MONUMENTS

During the cadastral survey, the land surveyor may have to set new monuments associated with the retracement of the existing right-of-way and at property controlling corners locations. Examples would be PC's, PT's, and locations having a change in the right-of-way width.

6.5.9.1 Existing Right-of-Way

All monuments set in conjunction with the retracement of the existing right-of-way shall consist of at least a 5/8" x 24" (15mm x 609mm) rebar and an aluminum cap not less than 2" (50mm). The cap will be stamped with the land surveyor's registration number and the year the monument was set. It is advisable to place a witness post with a Department decal as near to the monument as feasible, but it should be set between the monument and the PTW.

6.5.9.2 Property Controlling Corners

All found stones, posts, unmarked monuments and reestablished property controlling corners, shall be monumented with an approved Department aluminum monument. This monument consists of a 3 ¼" (82mm) cap attached to a 2 ½" x 30" (76mm x 760mm) flared aluminum pipe. A rebar should be placed inside of the aluminum pipe. When replacing an existing monument, such as a 1-inch pipe or stone, the new monument will occupy the position of the existing monument. The existing monument will be buried alongside the new monument and the corner recordation form will clearly state what was done with the existing monument.

Exceptions An exception is a found monument that meets the minimum requirements of ARM 8.94.3001(b) or a property controlling corner that would be set in asphalt. A property controlling corner that will be in asphalt may be monumented with a Department aluminum monument cut to a length of 24" (609mm), or a rebar and an appropriately stamped aluminum cap, or a bronze disk cemented into the asphalt. In lieu of a bronze disk, a Department aluminum monument may be cut to a suitable length and cemented into the asphalt. A bronze disk will require magnetic material set below the disk. In all cases, the top of the monument will be below the surface of the asphalt.

The cadastral request distributed by the Right-of-Way Bureau or District Right-of-Way Design may indicate a quarter corner is required. If this quarter corner is lost and cannot be established by any means other than a proportionate method, this corner does not need to be monumented. This exception does not apply to obliterated or existing corners. The relationship between this quarter corner and the section corners on each side must be shown on the certificate of survey. The monumentation of the section corners on each side of the lost quarter corner will conform to Department standards.

Reference Monuments All property controlling monuments that fall within any existing right-of-way will require, as a minimum, two reference monuments. If the surveyor believes the possibility exists that any property controlling monument may be subject to destruction, reference monuments should be established.

The relationship of all reference monuments to the property controlling corner will be shown on the corner recordation form. Reference monuments are typically a 2" (50mm) aluminum caps stamped RM with a number, for example "RM 1", a punch mark, the distance to the property controlling corner, the year, and the land surveyors registration number. Accessories to the corner, such as a scribed reference monument (a tree), and/or a witness post with a decal may also be set to aid in the location and perpetuation of property controlling corner.

If a corner recordation is being prepared for a BLM monument and the survey has not been approved by the BLM, reference monuments are required. The recordation must show these references and indicate the BLM survey has not been approved.

6.5.9.3 Set Monuments

Cadastral surveys consist of ties to found monuments as well as surveys required to set monuments at a calculated position. The final coordinates of found monuments will be based on the previous sections and these coordinates will be maintained in the final cadastral coordinate list. Coordinates of set monuments (calculated) will also be maintained in the final cadastral coordinate list. After a monument has been set, the coordinate position of that monument will be verified. This can be accomplished by obtaining additional measurements. For example:

- Set the monument.
- Change the backsight or the instrument location.
- Record necessary information to compute coordinates.
- Compute coordinates.
- Inverse to the calculated position.

The radial difference (based upon a horizontal inverse) between the calculated position and the actual set position must be less than or equal to 0.25 ft (0.076m). If the radial difference is greater than 0.25 ft (0.08m), additional survey will be necessary.

6.5.10 CORNER RECORDATIONS AND CERTIFICATE(S) OF SURVEY

Montana was one of the first states to enact a statute that addressed the perpetuation of property corners and property controlling corners. The Corner Recordation Act requires the filing of a corner recordation form for property controlling corner under certain circumstances. The land surveyor should refer to all applicable statutes and ARM and prepare and record all required corner recordations associated with the cadastral survey.

Department cadastral projects generally require a retracement and monumentation of the existing right-of-way and a dependent survey associated with property controlling corners. On large projects two certificates of survey should be prepared and recorded. One certificate would be associated with the retracement of the existing right-of-way; the other certificate would show the survey information associated with the property controlling corners. On smaller projects only one certificate of survey may be necessary.

Certificates of survey done by or for the Department that are based on the state plane coordinate system will include a state plane coordinate list containing the point number, north, east, and description. The units, datum and adjustment tag (NAD83/1999) must be included on the certificate of survey. The point number will also be placed next to the appropriate symbol on the drawing. The certificate of survey will indicate the Uniform Project Number (UPN); also know as the control number, the project identification, and project location.

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6.6 DELIVERABLES

At the completion of secondary traverses, radial surveys, and cadastral surveys the survey crew or the land surveyor will have generated several data collector files, coordinates, field notes, etc. All relevant electronic information should be placed in the appropriate project folder using the Department's Document Management System (DMS). All non-electronic records (original field notes, computation sheets, etc.) will be forwarded to the lead bureau.

Electronic survey information associated with a Consultant Design project will be provided to Consultant Design on a compact disk. In addition, all non-electronic (**original** field notes, computation sheets, etc.) survey information will be provided. Copies of the original field notes are not acceptable. All file names will conform to DMS file naming conventions. Consultant Design will forward this information to Photogrammetry & Survey Section for finalization and uploading to DMS.

6.6.1 PHOTGRAMMETRIC CONTROL AND CONTROL DENSIFICATION SURVEYS

The following files are required and will be uploaded to the appropriate folder using DMS:

- all raw data collector files
- final ASCII text coordinate list (format P N E Z)
- abstract (ASCII text file)
- all adjustment reports (horizontal and vertical)
- a read me file that explains contents off all files

6.6.2 CADASTRAL SURVEY

The following files are required and will be uploaded to the appropriate folder using DMS:

- all raw data collector files
- a final ASCII text coordinate list (all points observed format P N E D)
- a final ASCII text coordinate list (includes only information associated with the recorded certificate of survey format P N E D)

- all adjustment reports (generally horizontal only)
- as recorded certificate(s) of survey-MicroStation format
- as recorded corner recordations-optional
- a read me that explains the contents of all files.

In addition to the above, the Right-of-Way Bureau or District Right-of-Way Design will be provided paper copies of all recorded certificates of survey and corner recordations.

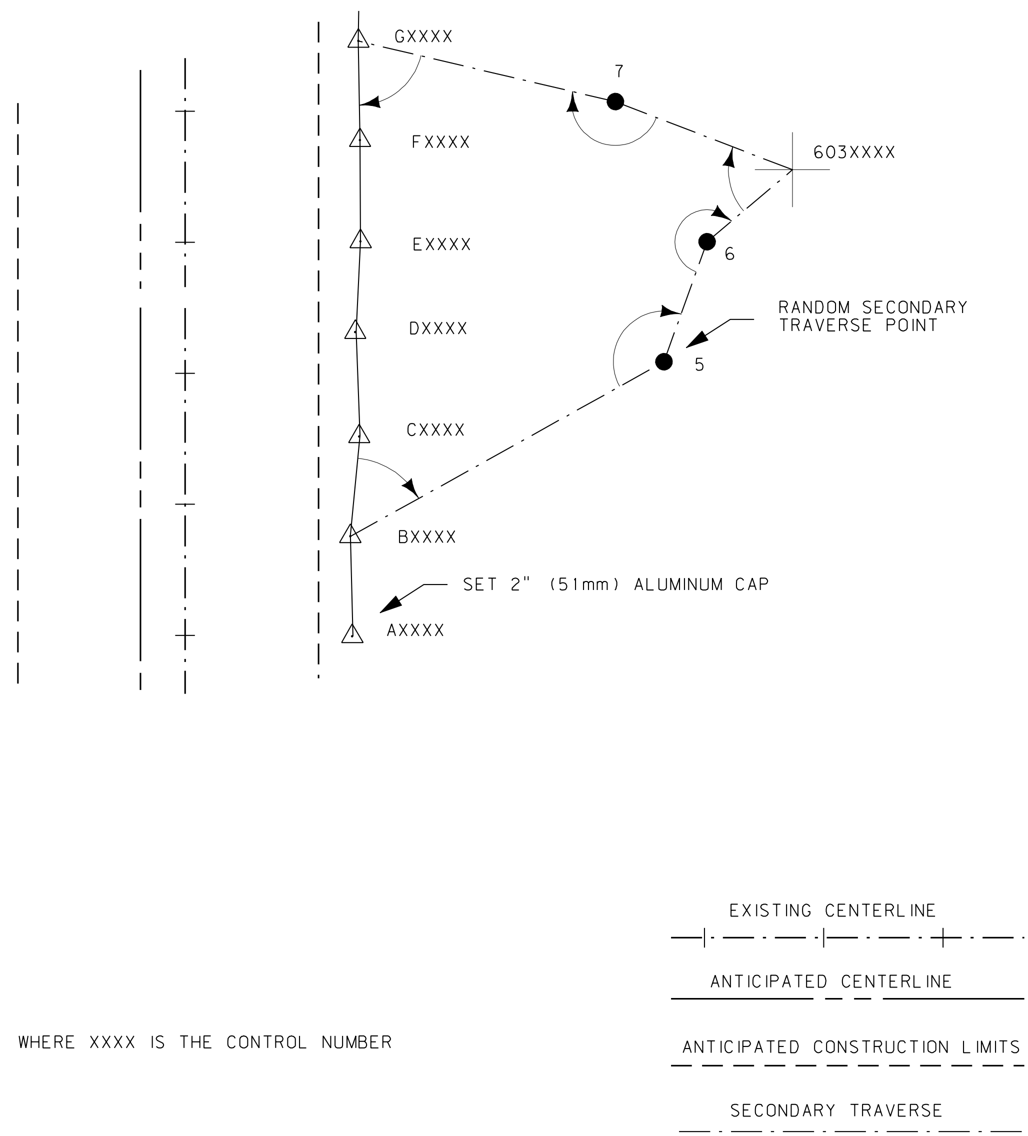

Figure 6-1
Secondary Traverse to a Photo Control Point

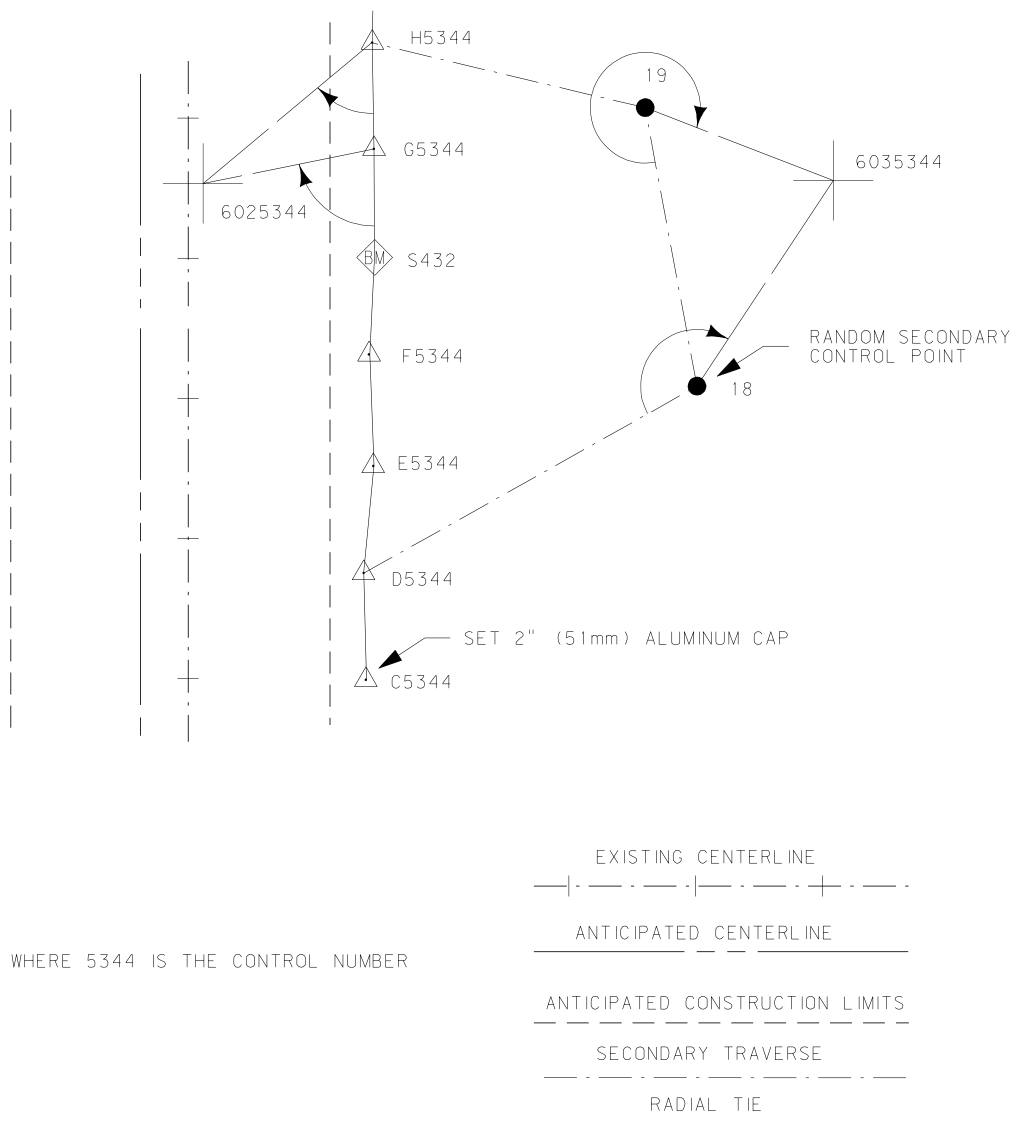

Figure 6-2
Radial Tie from a Secondary Traverse and from a Control Point

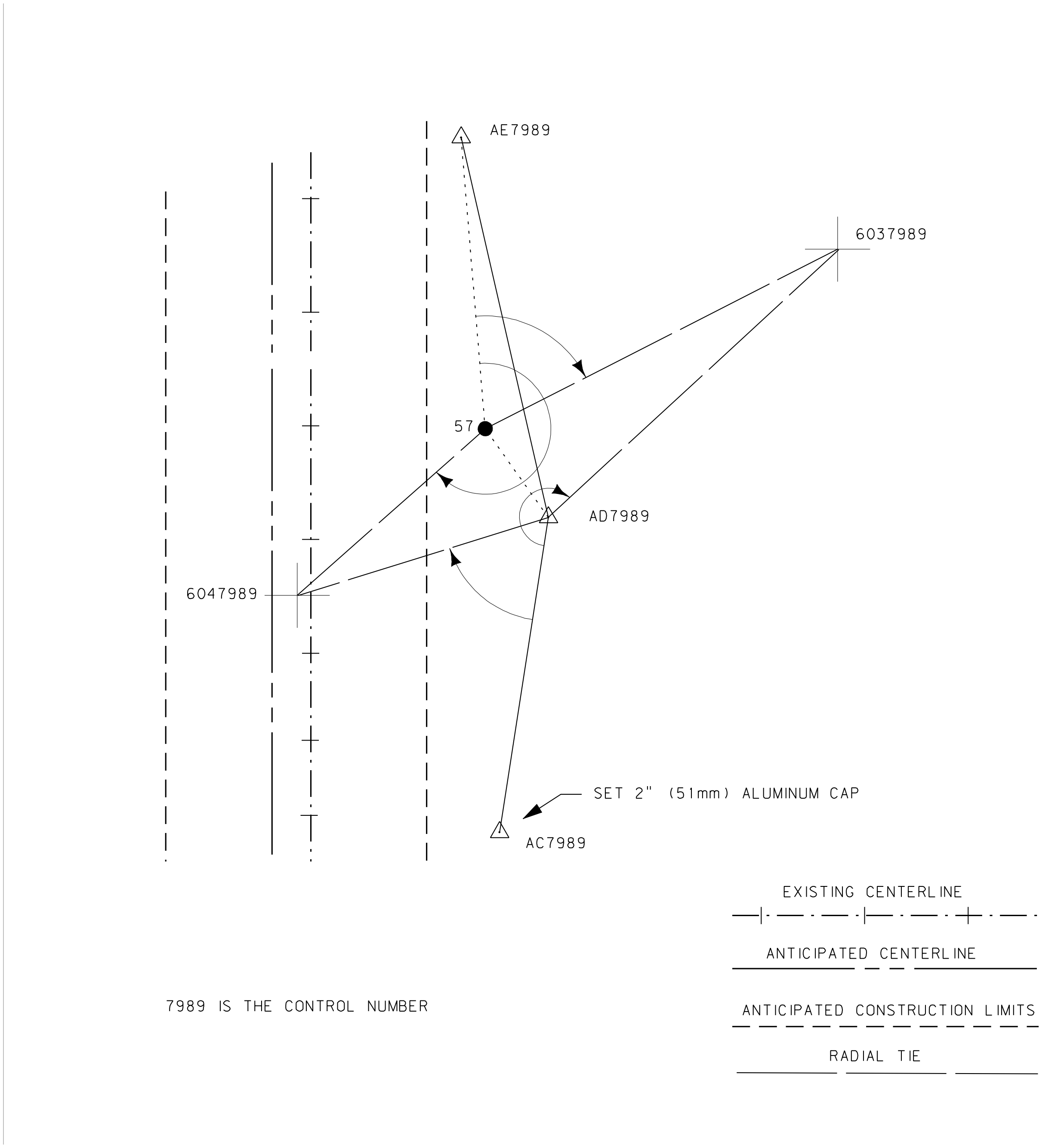

Figure 6-3
Radial Tie from a Control Point and a Random Point

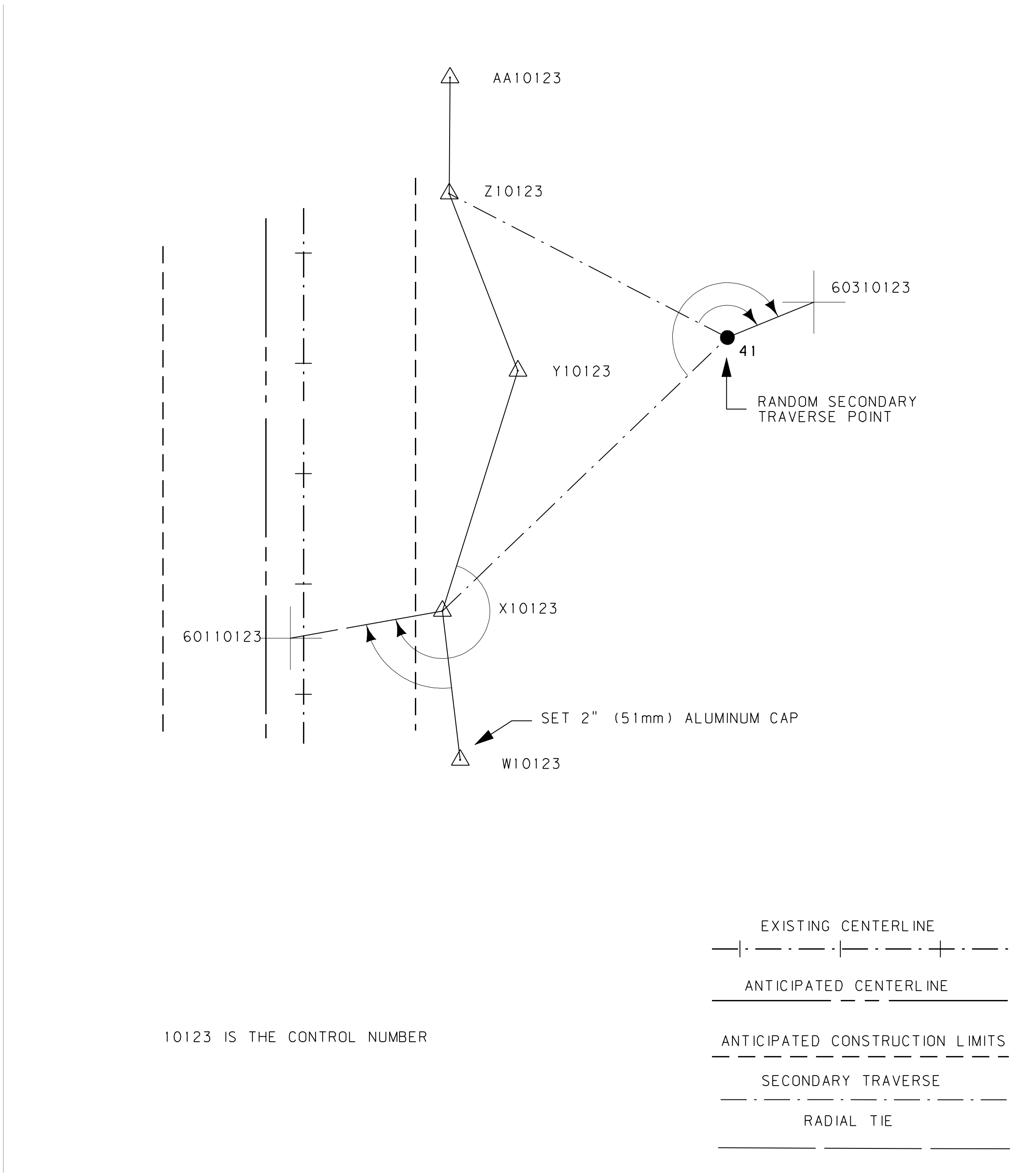

Figure 6-4
Radial Tie from a Control Point and a Secondary Traverse Point Using Different Backsights

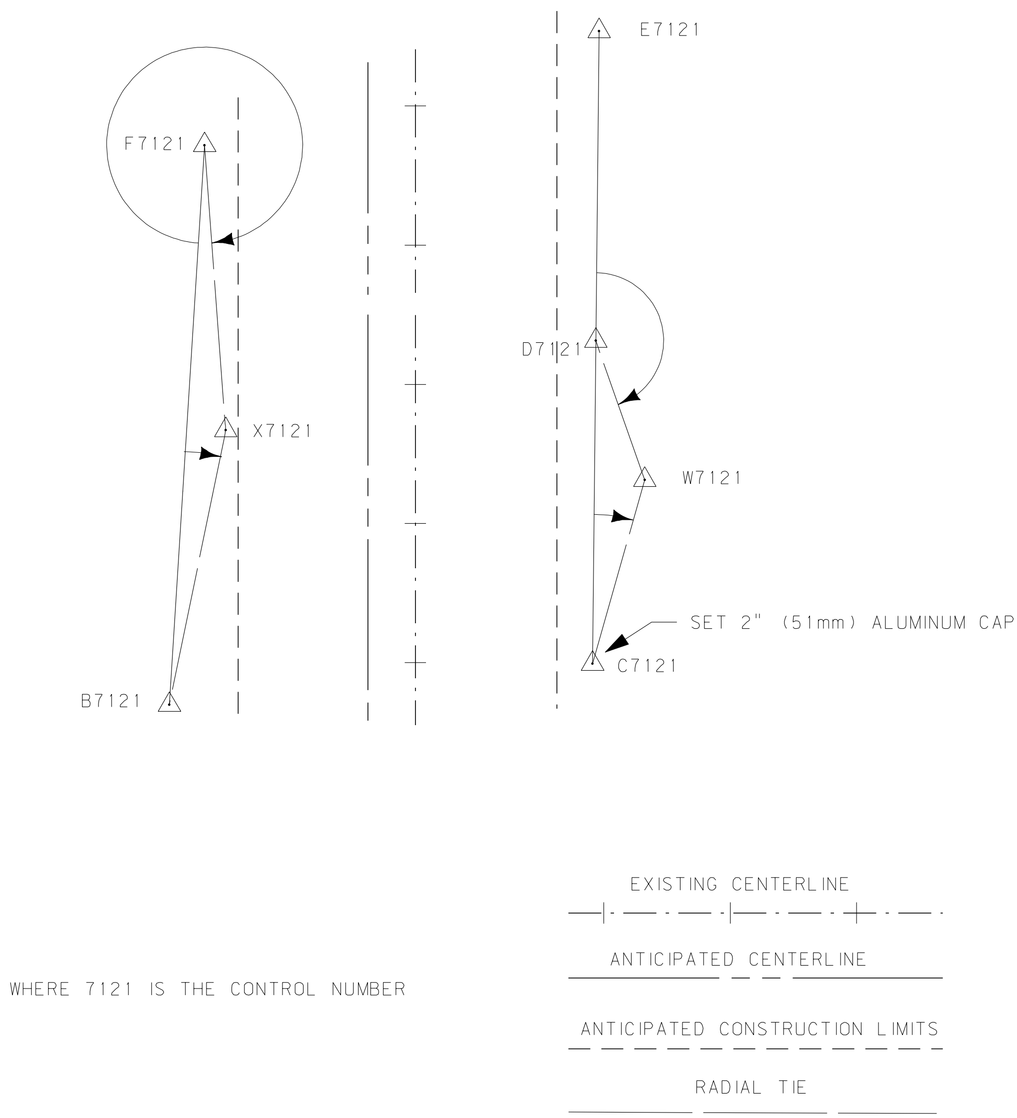

Figure 6-5
Radial Ties to Establish Additional Control Marks

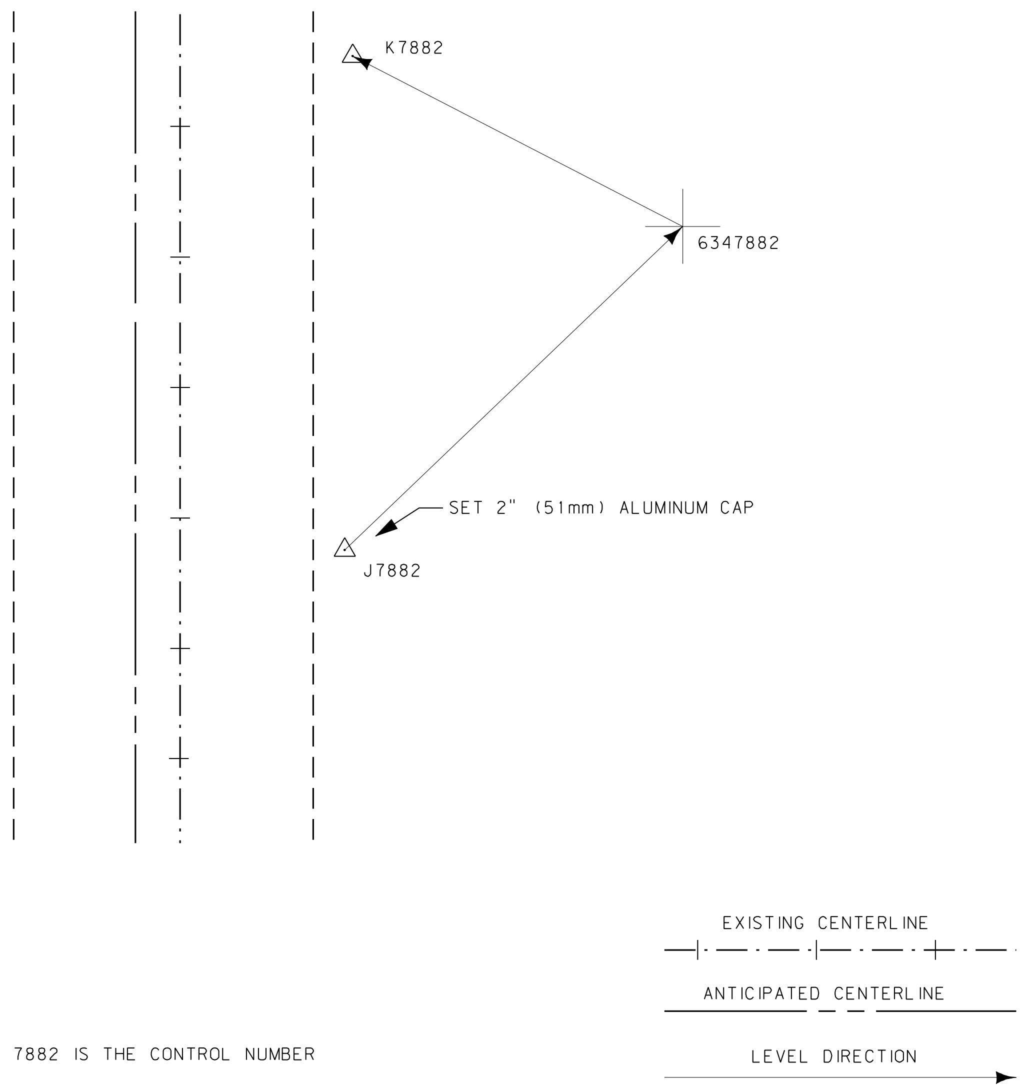

Figure 6-6
Differential Levels to a Photo Control Point

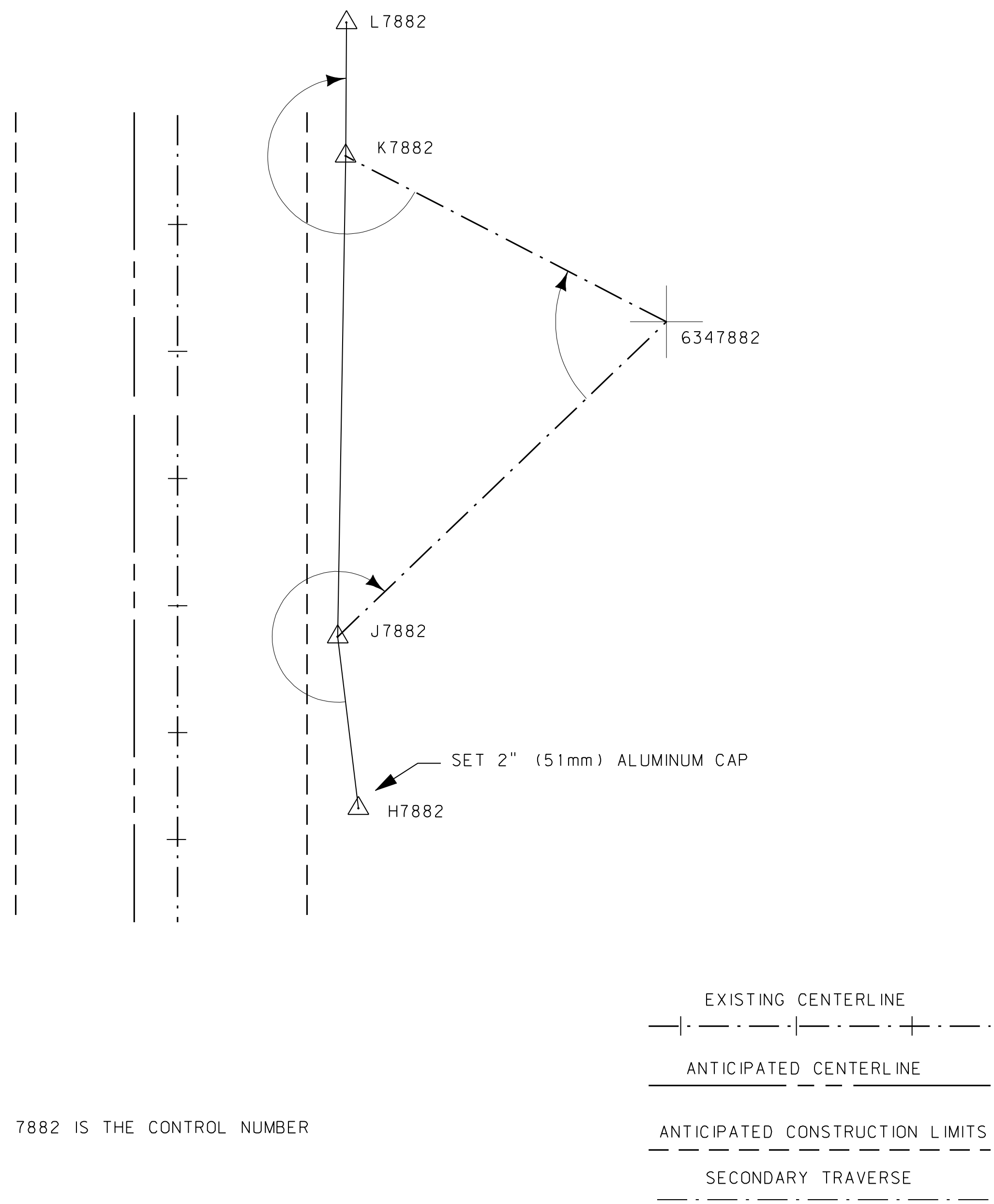

Figure 6-7
3D Traverse to a Photo Control Point

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Chapter 7

Reserved for Aerial Photography and Photogrammetry

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Chapter 8
Global Positioning System

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Chapter 8
Global Positioning System

This chapter deals with the Global Positioning System (GPS) surveys conducted by and for the Department. GPS is used by the Department to establish project control, cadastral (land) surveys, and resource grade data collection of miscellaneous features such as wetlands, core holes and signs. GPS positioning techniques (relative and differential) are continuing to evolve. As newer applications and procedures become available, the methods discussed in this chapter may be modified under certain situations. The Photogrammetry & Survey Section will evaluate and approve any modifications prior to implementation of GPS surveys associated with the establishment of project control and cadastral surveys. The Geospatial Imaging Unit, which is located within the Information Services Division (ISD), should be consulted regarding questions and procedures dealing with resource grade GPS data collection.

8.1 GPS CONTROL SURVEYS-GENERAL

The initial planning of a GPS control survey begins after the Preliminary Field Review (PFR) and the Survey Request has been distributed. The Survey Request will indicate the horizontal and vertical datum for the project.

Conventional control surveys were discussed in Chapter 5. Much of the information such as reference to the PFR, the Location Hydraulic Study Report (LHSR), research, reconnaissance, location of control points, numbering of control marks, monument specification, abstract requirements, etc., are also applicable to GPS control surveys. This information and specifications relating to conventional surveys will not be repeated in this chapter Refer to Chapter 5. However, differences between a conventional control survey and a GPS control survey will be noted in this chapter.

Consultant Design contracts require the consultant to submit a plan of their proposed GPS control survey to Consultant Design **prior** to commencing the control survey. This plan is forwarded to Photogrammetry & Survey for comments. The plan must include but is not limited to the following:

- horizontal and vertical control marks to be incorporated into the control survey
- receivers (model and manufacturer) and antennas (model and manufacturer) to be used

- proposed baselines to be observed
- horizontal and vertical datum
- coordinate units

8.1.1 RECEIVERS AND ANTENNAS

Survey grade receivers are required for all project control and cadastral surveys. Single or dual frequency receivers are acceptable. Single frequency receivers will not be used for base lines greater than 6.2 miles (10km). Receivers of different makes or models may be used on a given project. A minimum of three receivers is required, four or more receivers are preferable.

Geodetic quality antennas having a ground plane (either attached or built in) will be used for all GPS control surveys. It is recommended that different antenna types not be used on a project. If different antennas types are used, it must be recognized that each will have different phase center offsets, phase center variations, and different reference points for the measurement of antenna heights. The software used to process the baselines must be capable of accurately modeling all antennas used during the control survey.

8.1.2 SOURCES OF ERROR

As with all surveys, GPS surveys may contain instrumental errors, mistakes (blunders), systematic errors, and random errors. GPS surveys are subject to other types of errors and mistakes not found in conventional surveys. Examples are multipath, interference from high voltage power lines, chain link fences, incorrect antenna type used during processing of the data, incorrect method of measurement from the ground mark to the antenna, insufficient session length, etc.

Prior to commencing a GPS control survey; all tribrachs will be adjusted to assure accurate plumbing over the mark. The circular level vial will be checked and if necessary adjusted. All measuring devices used to measure the distance from the mark to the antenna will be checked for correct length. All tripods should be visually inspected for damage. All nuts and bolts should be tightened. Level bubbles on fixed height tripods and bipods should be verified and adjusted as required. Damaged or suspect equipment will not be used for any Department surveys.

All antenna heights will be measured **independently** in both feet and meters. The measured height will be recorded to the nearest 0.01 ft and the nearest 0.001m. The height of the antenna is measured in meters and this value is recorded. The height is then measured in feet and this value is recorded. The measured height in meters will **not** be converted and then recorded as the feet reading.

Immediately after both heights are recorded and prior to moving the antenna, a mathematical field check will be made to verify both recorded measurements agree. Dividing the metric height as recorded by 0.3048 makes the field check. This value is then compared with the recorded foot height. The difference must be less than or equal to 0.02 feet of the recorded foot measurement. If the difference is greater than 0.02 feet, both heights will be remeasured and field checked **before** the antenna is moved. The antenna will not be moved until both heights agree.

8.1.3 PROJECTION, DATUM, AND COORDINATES

A projection relates the spherical coordinates (latitude and longitude) on a curved surface (earth) to the corresponding grid coordinates (north and east) on a flat surface or a plane. By statute (Title 70, Chapter 22, Part 2), Montana uses the Lambert Conformal Projection.

A datum defines the size and shape of the earth as well as the origin and orientation of the coordinate system. Montana's state plane coordinate system is based on the North American Datum of 1983 (NAD83) and the Geodetic Reference System 1980 (GRS80) ellipsoid. The origin of this coordinate system and units are defined in state statutes. All state plane coordinate projects done by or for the Department will use only coordinates relative to NAD83. The use of coordinates relative to the North American Datum of 1927 (NAD27) was prohibited by statute and may not be used in Montana after July 1, 1993.

State plane coordinates as well as the corresponding geodetic coordinates relative to NAD83 have evolved over time. NAD83(1986) coordinates were derived from terrestrial observations and did not include any GPS observations. NAD83(1992) coordinates were based on GPS observations that were the basis of the original High Accuracy Reference Network (HARN) in Montana. The National Geodetic Survey (NGS) completed all observations and adjustments. NAD83(1999) coordinates, in Montana, are relative to the Continuously Operating Reference Stations (CORS), which provide the backbone of the National Spatial Reference System (NSRS). The preferred horizontal coordinate system for Department projects will be relative to NAD83(1999)/NAD83(CORS). However, NAD83(1992) coordinates are acceptable in certain situations. NAD83(1986) coordinates will not be used for any Department project.

Caution NGS data sheets may indicate coordinates are relative to NAD83(1992) or NAD83(1999). However, these coordinates as shown on the data sheet may not be acceptable. To determine if these coordinates are acceptable reference must be made to the method used to establish the mark. If the NGS data sheet indicates the geodetic coordinates were determined by classical geodetic methods, the geodetic coordinates of that mark will not be used for a Department project even though the data sheet indicates the coordinates are relative to NAD83(1992) or NAD83(1999). An example would be a triangulation station that was never occupied with GPS. The elevation may be valid provided it is third order or better.

NGS, at one time, made available a computer disk (CD) containing data sheets associated with their horizontal and vertical control. The information contained on these disks should not be used. The data may be out of date.

8.1.3.1 NAD83 vs. World Geodetic System of 1984 (WGS84)

Coordinates for Montana, as well as the rest of North America including Canada, are based on a common datum known as NAD83. A horizontal datum is defined by eight constants. Three define the origin of the coordinate system, three define the orientation of the coordinate system, and two define the dimensions of the reference ellipsoid. The defining parameters of NAD83 are based on the reference ellipsoid known as GRS80. The eight defining constants for GRS80 have not changed since they were originally defined. The origin, orientation, and shape and size have remained the same. On the other hand, the defining parameters of WGS84, though similar to those of GRS80, have changed through time. Most notably, the defining constants for the origin of WGS84 have changed. As the understanding of the position of the Earth's center of mass has improved the origin of the WGS84 reference ellipsoid has been changed to closely approximate that updated location. Currently, there have been four different defining origins for WGS84 over the years and coordinates based on WGS84 are not consistent between these changes in origin.

The origins of NAD83 and WGS84 were very close at the inception of the WGS84 reference ellipsoid but over the years the WGS84 origin has moved apart from the NAD83 defined origin by more than two meters. This translates to more than a **meter difference** in the coordinates between the two systems for the same point on the ground. The legislated coordinate system for Montana is NAD83 and remains a common, consistent coordinate system. The WGS84 reference system, though consistent with today's interpretation of the Earth, changes over time creating different coordinates for the same point between each change.

8.1.4 ELEVATIONS

Elevations associated with state plane coordinate projects will be relative to North American Vertical Datum of 1988 (NAVD88).

8.1.5 GPS PROJECT CONTROL POINTS

The standard control monument is a 5/8-inch x **30-inch** (16mm x 763mm) rebar with an approved 2-inch (51mm) aluminum cap. The spacing between marks should not be greater than 1000 to 1200 ft (300m to 360m). This spacing facilitates the collection of topography using data collectors, additional surveys, and ultimately the staking of the designed project. Adjacent marks must provide for visibility both forward and back.

As opposed to a conventional control survey, GPS control surveys do not require control marks along both sides of the Present Traveled Way (PTW). Instead, control marks are set at strategic locations through the project. Terrain will generally dictate the spacing and location of the control marks. The PFR should be reviewed to identify possible shifts in the existing alignment. If the PFR indicates the alignment will shift westerly, the control marks should be placed easterly of the existing PTW. If permission to enter private lands has been granted, the control marks can be placed outside the existing right-of-way. In all cases the new control marks should be established outside the anticipated construction limits.

Figure 8-1 depicts a portion of a GPS control survey configuration associated with a reconstruction project. NGS bench mark S452 will be occupied by GPS and can be used for additional preliminary surveys and staking of the project. The location of this mark may eliminate the need for a project control mark in this area. In no cases will the NGS bench mark be assigned any other designation other than S452.

Segments of a project may not be suitable for GPS. An example would be dense forested areas. In these cases, at least two marks (azimuth pair) will be set near each end of the area(s) having obstructions. The distance between an azimuth pair should be established at the maximum practical distance. This distance, if possible, should not be less than 1000 ft (300m). A conventional control survey is then used to establish control between the GPS azimuth pairs. Refer to Figure 8-8.

Figure 8-2 shows a possible configuration of the placement of control marks associated with a bridge project that will be mapped by aerial photography. Control marks B8998 and D8998 were set to assist in the collection of data required by hydraulics. In order to utilize a total station, for the collection of topographic features, a backsight must be available at all marks. This figure also shows photo control marks required for aerial mapping. Since photo control mark 6038998 is located near the PTW it may be used for additional preliminary survey and staking of the project. Therefore, an additional control mark in the vicinity of this photo control mark generally would not be required. In no case will this mark be assigned any other designation other than 6038998.

After the control has been established additional control marks and/or additional temporary points for topographic data collection may be required. These additional marks can be established using conventional instruments (Chapter 6) or GPS methods per this chapter.

8.1.6 ABSTRACT

An abstract is required. An abstract is a text file that contains the location and description of all marks associated with a GPS control survey. It is advisable all information required to create the final abstract be recorded in a set of field notes. This information should be recorded at the time the monument is set or at the time the existing monument is found. The information associated with the point abstract should consist of the mark name (point ID which is the same as the mark name as stamped on the cap), if the monument was found or set, size and type of monument, information associated with location (how to reach), reference post (RP), distance and direction from the PTW, the location of witness post relative to the monument, and the location of the monument in relationship to nearby features such as fences.

A photo control mark (wing point) that is not located near the PTW and will not be used for data collection or construction does not require a how to reach description, but all other information such as type of monument will be included in the abstract associated with the wing point. The abstract associated with a wing point that will be occupied by conventional surveys must include a how to reach description. Refer to Form 5-1.

8.1.7 RECONNAISSANCE

Reconnaissance consists of a field check of the proposed primary horizontal and vertical control marks that may be used during the control survey. These primary control marks are generally HARN marks, previous project control marks, or NGS and United States Geological Survey (USGS) bench marks. The main purposes of the reconnaissance are to verify the mark exists, that it is suitable for GPS observations (free from obstructions, interference, possible multipath), that it is stable, to gather information for the control point abstract, and to complete required obstruction diagrams.

8.1.7.1 Obstruction (Station Visibility) Diagrams

Obstruction diagrams must be completed prior to the GPS survey for all points having obstructions higher than 15 degrees. Survey points without obstruction should be noted or an obstruction diagram may be completed indicating no obstruction exist above 15 degrees. Refer to Form 8-1 for a sample obstruction diagram. See Appendix A for an example of a completed obstruction diagram. The obstruction diagram should include the following:

- Station ID
- Department's control number
- Date
- Declination set in hand compass
- Name of person completing the obstruction diagram
- Any relevant comments

8.1.8 SCHEDULE OF OBSERVATIONS

Observations should be scheduled during optimal satellite geometry. Items to be considered include:

- Number and elevations of satellites
- Obstructions
- Position Dilution of Precision (PDOP)
- Vertical Dilution of Precision (VDOP)

8.1.9 OBSERVATION LOG

Field notes are required and may be recorded in a standard field notebook or recorded on special forms. Form 8-2 is a sample static/fast static (rapid static) observation log. See Appendix A for an example of a completed static/fast static log. This form may be used for a session that includes only one occupation (a base receiver) or for a session that includes multiple occupations (a roving receiver) made during a fast static session. A separate observation log is required for each session. All occupations will include all required information. The page number information must be completed. Information recorded in field notes or observation logs will include:

- Department's control number and project number (Project ID)
- date of observation
- receiver number (serial number or inventory number)
- receiver manufacture and model number
- file name
- observer name
- antenna manufacture and model number
- height of antenna, method of measurement, and reference point measured to on the antenna (typically slant height to the ground plane or vertical height to the base of the antenna)
- mark name (Station ID/GPS station name) observed, description, and rubbing
- starting and ending time of the observation
- notes regarding any problems or edits that need to be made

The mark name in all GPS files will agree with the stamped mark name. For example, if the observed mark is F5675, then the name of the mark as input into the GPS file will be F5675. A mark name such as CP1 or 1 is not acceptable.

Antennas attached to a tripod and tribrach requires the measurement of the antenna height at three different locations (approximately 120 degrees apart) around the ground plane. The three measurements should agree within ±0.01 ft (0.003m). Readings outside these limits indicate an antenna that has not been leveled correctly, the tribrach requires adjustment, and/or the ground plane may be warped. Excessively warped ground planes may not be used for Department control surveys.

8.2 PROJECT CONTROL

Project control consists of all marks occupied during the control survey. These marks include, but are not limited to the following:

- NGS horizontal control marks (HARN/CORS)
- NGS and/or USGS vertical control marks (bench marks)
- MDT horizontal and/or vertical control marks from previous projects
- Newly established project control marks
- Photo control marks (center and wing points)

8.2.1 HORIZONTAL CONTROL

Department GPS control surveys require connection to a minimum of three known horizontal marks. These three marks can have coordinates relative to either NAD83(1992) or NAD83(1999)/NAD83(CORS). The three marks must be based on the same adjustment tag and will be located in three quadrants relative to the limits of the project. NAD27, NAD83(1986), or WGS84 coordinates will not be used for any Department projects, nor will these coordinates be transformed to a NAD83(1992) or a NAD83(1999)/NAD83(CORS) coordinate. Information associated with the HARN marks can be obtained from NGS. Information regarding previous Department projects can be obtained from the Photogrammetry & Survey Section in Helena. Online Positioning User Service OPUS solutions may be used in certain situations as discussed in this chapter.

Depending upon the location of the project relative to the known control, three general situations exist:

- connection to only the HARN
- connection to the HARN and/or previous Department projects
- connection to CORS, HARN, previous project marks, and/or OPUS solutions

8.2.1.1 HARN Marks Only

Refer to Figure 8-3. In this figure Taylor, F204, and H570 represent HARN marks. It is recommended redundant (a second independent) baselines be obtained from the

HARN marks to the new project control marks. The use of a single baseline to the existing HARN and a minimally constrained horizontal adjustment (latitude and longitude held at one mark) may indicate an unacceptable coordinate comparison at one or both of the other HARN marks. In this situation additional observations may be required since it is not possible to determine if the horizontal coordinate comparison is a result of an inferior baseline solution or the HARN mark has been displaced. The time and cost of the additional observation may greatly exceed the time had redundant observations been made at the time of the original connections.

The use of either the precise or the rapid ephemeredes (orbits) should be considered for the processing of the longer baselines.

The final coordinates of all new control marks will be based on one fully constrained (horizontal and vertical) adjustment. The horizontal position (geodetic) of all HARN marks, and whatever vertical deemed acceptable will be constrained. This network adjustment will consist of all acceptable static and/or fast static observations associated with the project network. The baselines associated with the connection to the HARN marks will not be adjusted first, and then followed by a second network adjustment of the project control marks.

8.2.1.2 Combination of HARN and/or Previous Department Project Marks

This combination of control marks is essentially identical to the previous explanation. Refer to Figure 8-4. In this figure, A, B, and C represents either a HARN mark or a previous MDT project control mark. The Photogrammetry & Survey Section in Helena maintains coordinates associated with previous MDT projects. In addition, Trimble™ processed vectors (SSF or SST) are also available. If the coordinate of a control mark(s) will be used from a previous Department project, the selected mark should be a monument such as a NGS bench mark. The typical project control mark [5/8 inch x 30 inch (0.016m x 0.762m) rebar and aluminum cap] is not as stable. If a previously processed vector is used, it is advisable to select a mark that had a direct observation from an existing HARN mark. The horizontal datum associated with the three control marks must be compatible with the other control marks. Mixing of NAD83(1992) and NAD83(1999) is not acceptable.

Project network adjustment will be identical to a network adjustment that utilizes HARN marks exclusively.

8.2.1.3 Combination of CORS, HARN, Previous Project Marks, and/or OPUS

Depending upon the location of a project relative to existing control (HARN and/or previous Department control marks) connection to three existing marks may not be readily available. Refer to Figure 8-5 (A). In this figure, existing control is available easterly and southwesterly of the new project. These marks are designated A and B. In lieu of a direct connection to the third existing control mark as shown in Figures 8-3 and 8-4, the coordinates for this third control mark, (C), may be determined by:

- A network adjustment using CORS data [Receiver Independent Exchange Format (RINEX)] or
- OPUS solutions

The use of CORS or OPUS dictates the coordinates of the project will be relative to NAD83(1999)/NAD83(CORS). Point C should be located outside the project limits and should be a more substantial mark than the typical Department control mark (rebar and aluminum cap). A first or second order NGS bench mark is preferred.

Network Adjustment-and CORS Data (RINEX) The geodetic coordinates of mark C may be determined based on a fully constrained network adjustment that constrains a minimum of three CORS. This adjustment requires two independent occupations of C, CORS data, and the use of either the rapid or precise ephemeris (orbits). RINEX files and the ephemeris may be obtained from NGS. Refer to Section 8.4 and 8.5 for additional information and suggestions.

OPUS Currently two methods are available using OPUS to determine geodetic coordinates of C. The first method will mean the geodetic coordinates of two or more individual OPUS solutions. If a mean value of C is to be determined from two or more OPUS solutions, each OPUS solution should meet the recommended quality parameters as suggested by NGS. These parameters are currently:

- at least 90% or more of the observations are used
- at least 50% of the ambiguities should be fixed
- overall root mean square (rms) should seldom exceed 0.03m (0.10 ft)
- peak to peak errors should seldom exceed 0.05m (0.16 ft)

A mean value from an OPUS solution is acceptable provided the difference in the north and the east coordinate is ≤0.05 ft (0.015m) and the difference in the elevation is

≤0.07 ft (0.021m). The OPUS solutions should use either the precise or rapid ephemeris data. The ultra rapid ephemeris should not be used.

The second method to obtain the geodetic coordinates of C, makes use of the G files, which are included in the extended output of an OPUS solution and a fully constrained network adjustment. This adjustment requires two independent occupations of C. Either the rapid or precise ephemeredes may be used. Refer to Section 8.4 and 8.5 for additional information and suggestions.

Projects that incorporate a network adjustment for the determination of geodetic coordinates at C (CORS data or G files) require two fully constrained network adjustments. The first network adjustment consists of only the GPS observations required to obtain the geodetic coordinates of point C. Once this network has been successfully adjusted, the second network adjustment will constrain the geodetic coordinates of A, B, and C and will consist of all baselines associated with the project.

8.2.2 VERTICAL CONTROL

The vertical datum for all state plane coordinate projects will be relative to NAVD88. Information regarding NGS bench marks can be obtained from their web site. Information regarding United State Geological Survey (USGS) bench marks can be obtained from their Denver office. USGS bench marks are relative to the National Geodetic Vertical Datum of 1929 (NGVD29) and must be updated to NAVD88. Three general methods may be used to update NGVD29 elevations to a NAVD88 elevation

The first method uses NGS's program Vertical Conversion (VERTCON). The second method is to apply a vertical shift to the published elevations of the USGS bench marks. In order to do this, at least one bench mark in the area having both a published NGVD29 and a published NAVD88 elevation is required. The NAVD88 elevation of the other USGS bench marks is then obtained by applying this vertical shift to the published NGVD29 elevation. The accuracy of this method will depend partly upon the distance the USGS bench mark is from the bench mark having both a NGVD29 and a NAVD88 elevation. The third method requires two bench marks having both a NGVD29 and a NAVD88 elevation. The vertical shift at each mark is determined. This vertical shift is then distributed in proportion to the distances between the individual USGS bench marks. All methods to convert NGVD29 elevations to NAVD88 elevations are estimates at best and should be used with caution.

Most Department projects will require a minimum of four vertical marks (bench marks). There are exceptions. If fewer than four vertical marks are to be used, prior approval must be obtained from the Photogrammetry & Survey Section. The location of the bench marks should be strategically located in relation to the project. Larger projects, such as a reconstruction, will require additional vertical marks. As a general rule, all bench marks located within one mile (1.6km) of the project will be occupied and included in the GPS network.

8.2.2.1 Temporary Bench Marks (TBM)

Bench marks that would normally be incorporated into the GPS control survey but cannot be occupied directly with GPS, requires the establishment of a TBM. An example would be NGS bench marks located in head walls, rock outcrops, or a bench mark having excessive obstructions. The elevation of the TBM is determined by differential levels from the NGS bench mark to the TBM and back to the NGS bench mark. All TBMs will be included in the control abstract. The abstract will include all required information and will also include the location of the TBM relative to the original bench mark. The TBM will consist of a 5/8 inch x **30-inch** (0.016m x 0.76m) rebar, 2” (51mm) aluminum cap, and a witness post. The aluminum cap will be stamped with the original bench mark name, TBM, and the year. For example, NGS bench mark S234 is located in a bridge end and cannot be occupied. The TBM would be set and stamped S234TBM and the year. An alternative to establishing a TBM is to run differential levels from the NGS bench mark to the nearest project control mark and back to the NGS bench mark.

Caution The maximum vertical closure associated with the establishment of the elevation for a TBM must be equal to or less than 0.02 ft (0.006m). The Department has many TBMs that were set in conjunction with previous projects. Prior to setting a new TBM, contact Photogrammetry & Survey to determine if a TBM already exists in the vicinity of the NGS bench mark.

8.2.2.2 Project Elevations

A fully constrained 3D adjustment will yield latitude, longitude, and a GPS derived orthometric height (elevation) of all occupied marks in the network. For some projects, GPS derived elevations may be deemed accurate enough for the intended scope of the project. Other projects may require an independent confirmation of the GPS derived elevations. The following are suggested guidelines:

- Urban areas (curb and gutters) - differential levels should be used to verify GPS derived elevations.

- Bridge projects – differential levels, as a minimum, are recommended between control marks that will be used for staking the structure.

- Rural project – differential levels and/or reciprocal trigonometric levels as required to verify GPS derived elevations.

The above guidelines should not be construed so as to be totally controlling. Differential levels and/or reciprocal trigonometric levels can be used at anytime and for any project, if additional verification of the GPS derived elevations are required.

8.3 NETWORK DESIGN-GPS CONTROL SURVEYS

At least three known horizontal control marks must be incorporated into all Department GPS control surveys. These marks may consist of any combination of OPUS solutions, CORS, HARN marks, or existing Department control marks. The number of known vertical marks to be incorporated depends upon the scope and length of the project. A minimum of four known vertical marks is generally required.

8.3.1 GENERAL

Each observed mark will be connected with an independent baseline to at least two other marks in the network. This does not mean as planned, but as adjusted in the network. A baseline cannot be excluded from a network adjustment if that mark has only two independent baselines associated with it. If it is necessary to exclude one of the two baselines, then an additional occupation of that mark is required.

The purpose of an independent observation is to isolate and identify set up errors, antenna height measurement errors, and multipath. If two base receivers are being used and a rover occupies a mark, the processing of a baseline from each base to the rover position does not constitute an independent observation. A common baseline (redundant observation) must exist from session to session. The sessions do not have to be sequential. If a receiver is to remain at a mark during back-to-back sessions, an independent setup is required (the height of the antenna changed) between the end of the first session and the start of the second session. Fixed height poles must be re-plumbed.

8.3.2 DIRECT OBSERVATIONS BETWEEN ADJACENT CONTROL MARKS

Adjacent marks should be connected by a direct observation in certain situations. Refer to Figure 8-7. If the distance to a mark is less than 10% of the sum of the total distances to that mark and the adjacent mark, then a baseline should be observed between the marks. Assume the baseline length from B10020 to NGS bench mark S497 plus the baseline length from B10020 to Y10020 is 2000 ft (609m). If the distance between Y10020 and S497 is less than 10% or 200 ft (61m) a baseline should be obtained between Y10020 and S497.

A minimum of one independent observation (processed baseline) must be made between all azimuth pairs.

8.3.3 NETWORK CONFIGURATIONS

Network configurations typically do not have a significant affect upon the final adjusted horizontal coordinates. However, the configuration and location of vertical marks can have a significant affect upon the final adjusted elevations. Acceptable network configurations provide for detection of systematic errors and excessive random errors, mistakes, and providing adequate ties to existing NGS or Department control marks. The scope and limits of a project may determine the network configuration. Two general network configurations are acceptable, and will be referred to as loop or radial. It is recognized that in most cases the GPS project network will not conform strictly to one or the other. The final network may be a combination of both configurations.

8.3.3.1 Loop Configuration

The loop configuration is similar to a traverse. Certain restrictions apply. Refer to Figure 8-6. Assume four receivers are being utilized. The survey begins with receivers being placed at A7777, B7777, C7777, and F7777. After sufficient data has been acquired, data is available to process six baselines. However, there are only three independent baselines. Assume, the baselines from A7777 to B7777, from B7777 to C7777, and from C7777 to F7777 are selected for processing. This does not meet the requirements that all observed marks are to be connected to at least two other marks in the network using independent observations. The two baselines from B7777 to the adjacent marks and the two baselines from C7777 to the adjacent marks are not independent baselines. The baselines between the adjacent marks must be observed at another time, or an additional baseline must be observed to each of the four marks at a different time during the control survey. The purpose of an independent observation is to isolate set up errors, antenna height measurement errors, and identify the possible affect of multipath.

8.3.3.2 Radial Configuration

Refer to Figure 8-7. Points A10020, B10020, and C10020 represent project control marks that will be occupied by a reference (base) receiver while two or more receivers rove through the remaining project control marks.

Assuming three receivers are utilized, the reference receiver would occupy A10020 and the indicated radial observations would be completed. Prior to moving the reference receiver from A10020 to B10020, the rovers will occupy B10020 and C10020. The reference receiver is then moved to B10020 and the radial observations completed at B10020. Prior to moving the reference receiver from B10020 to C10020, the rovers would occupy A10020 and C10020. The reference receiver is then moved to C10020 and the radial observation completed at C10020. Prior to moving the reference receiver, the rovers would occupy A10020 and B10020. This results in at least two independent observations between all marks observed.

Caution This configuration is not valid if two simultaneous operating reference receivers are used (at A10020 and B10020) and a baseline is processed from each reference to the rover. The baselines from each reference receiver to the rover are not independent baselines.

8.3.3.3 Excessive GPS Obstructions

There may be areas on a project where GPS may not be successful due to excessive obstructions. Examples of obstructions associated with rural projects are heavily forested areas and steep canyons. Obstructions in urban areas could be buildings, tree lined streets or high chain link fences near control marks. Multipath may also become a problem in urban areas.

In some situations it may be obvious that GPS will not be successful. In questionable areas, the decision that GPS is not useful due to obstructions should not be made unless detailed obstruction diagrams are available and the obstruction diagrams have been incorporated into planning software.

Excessive obstructions may occur in isolated areas within a project, or these obstructions may occur in large areas within the project limits. In either case an azimuth pair should be established near each end of all obstructed areas. The azimuth pair does not need to be adjacent to the Present Traveled Way (PTW) but should be outside the estimated construction limits. An independent baseline will be observed between each mark in an azimuth pair. Each mark associated with an azimuth pair must be inter-visible and will be at least 1000 ft (300m) apart.

Refer to Figure 8-8. Figure 8-8(A) indicates the situation where an isolated area of obstructions exists. GPS observations would be used to establish coordinates for all project marks except for those in the obstructed area. The azimuth pairs (M8765 and N8765, and P8765 and R8765) in conjunction with a conventional control traverse, including differential levels, would be used to establish the coordinates of the control marks between N8765 and P8765. The angles shown are required. It is important that control mark names are not duplicated.

Figure 8-8(B) indicates the situation where a large segment of a project is unsuitable for GPS observations, but it was possible to locate the azimuth pair G5678 and H5678 within the obstructed area. Two conventional closed traverses, including differential levels, would be used to establish the coordinates of the control mark in the areas having obstructions. The first traverse would start at F5678 with a backsight to E5678. This traverse would close on G5678 with a closing angle to H5678. The second traverse would start at H5678 with a backsight to G5678. This traverse would close on J5678 with a closing angle to K5678.

If it were not possible to establish the azimuth pair G5678 and H5678, one conventional traverse would be required. This traverse would commence at F5678 with a backsight to E5678. This traverse would close on J5678 with a closing angle to K5678.

Refer to the Chapter 4 regarding maximum closures and Chapter 5 regarding general requirements and adjustments associated with conventional control surveys.

8.4 DATA COLLECTION-GPS CONTROL SURVEYS

Data must be collected using survey grade receivers. Raw data may be logged to either a receiver or a data collector. It is recommended data be collected at 15 second intervals and an elevation mask of 15°. The PDOP should not exceed six (6) during the collection of data. The GPS name of the mark (mark ID), as stored in the raw data file will be identical to the stamping on the monument. For example, mark R12345 is observed. The mark name in the GPS file will be R12345. No other mark identification name is acceptable. The same mark ID will be used regardless of the number of times that mark is occupied during the GPS control survey. Original field notes will be provided for all marks occupied during the GPS control survey. The field notes may be recorded in standard field books or on special forms. Refer to Form 8-2 and an example of a completed Form 8-2 in Appendix A.

8.4.1 OBSERVATION TIMES

Observation times at a mark, to a certain extent, are a function of baseline length. Various methods can be used to determine occupation times. Table 8-1 indicates suggested occupation times for dual frequency receivers for various baseline lengths.

Baseline distance	Number of SV's	Time (minutes)
Less than 20km (12.4 miles)	4	20
	5	15
	6 or more	9
20 to 35km (12.4 to 21.7 miles)	4	40
	5	30
	6 or more	16
35 to 60km (21.7 to 37.2 miles)	4	60
	5	45
	6 or more	30
Greater than 60km (37.2 miles)	4	180
	5	135
	6 or more	95

Table 8-1
Suggested Occupation Times

Static or fast static observations that utilize the CORS may require observation times greater than shown above. Observations submitted to NGS for an OPUS solution should adhere to their recommended observation times.

An absolute minimum of four common satellites is necessary to generate an acceptable baseline. GPS baseline observations associated with Department projects should have a minimum of five satellites for at least 75% of the occupation. Baselines may be observed with four common satellites for 25% of the occupation, provided the PDOP does not exceed seven (7).

8.4.2 SESSION OUTLINE

Based on the actual observations, a report (text file) will be created showing receiver file names, marks occupied, and the start/end time of the observation. Refer to Form 8-3. The session outline may be different than the proposed observing schedule due to scheduling problems, or equipment failures.

8.5 NETWORK ADUSTMENT-GPS CONTROL SURVEYS

All GPS surveys associated with project control will be adjusted using either GPSurvey™ (Version 2.35 Web Patch 2.35a) or the latest version of Trimble Geomatics Office™ (TGO).

8.5.1 RAW GPS FILES

All unedited GPS raw files will be included as part of the required data. By default GPSurvey™ and TGO™ saves this information to a project folder.

Information contained in all raw GPS files must be verified. Data verification consists of confirming the information in the raw GPS files. Trimble™ refers to this as “checkin”. During the “checkin” process, the field notes must be available. The data in the receiver or the data collector file must be compatible with the information recorded in the field notes. Information that must be verified includes mark name, height of antenna (method of measurement and to what part of the antenna), start and stop times, and type of antenna. The height in meters must be verified by a conversion of this metric value to the foot value and compared with height as recorded in the field notes.

The original field notes must contain both units as measured and recorded in the field. The field notes may also indicate problems encountered by the observer or notes indicating input errors made at the time of occupation such as wrong mark name (e.g. W12345 was stored in the GPS file instead of the correct mark ID which was X12345).

Data collected using receivers other than Trimble™ require RINEX Version 2.XX files. The standard measurement height for RINEX files is to the bottom of the antenna mount. The bottom of the antenna is also referred to as the antenna reference point (ARP). All observation and any required navigation files will be included as part of the required deliverables. A RINEX file will not contain multiple occupations. After the “checkin” of a RINEX file a RNX file will be created by the software. All RNX files will be included as part of the required deliverables.

8.5.2 BASELINE PROCESSING

The coordinates of the initial mark from which baselines are to be processed (coordinate seeding) should be a HARN mark, a previous Department control mark, or an OPUS solution. An initial positional error estimate of 30 feet will result in 1 part per million (ppm) error in the computed baseline. The use of the broadcast ephemeris is acceptable in some cases, but the use of either the rapid or precise ephemeris may be beneficial for the processing of longer baselines.

As a general rule, only independent baselines should be processed and included in the network adjustment. Trivial baselines may be processed, but should not be included in the network adjustment. Assuming four receivers are used during a session, a total number of possible baselines in that session would be equal to 3+2+1=6. The total number of possible baselines can also be determined by N(N-1) / 2 where N is equal to the number of receivers used in that session The number of independent baselines would be equal to 3 and can be determine by the equation N - 1. Where N is equal to the number of receivers in that session.

Final coordinates as produced in a least squares network adjustment that includes all baselines should be substantially identical to the coordinates generated in a network adjustment that contains only independent baselines. The inclusion of trivial baselines produces an overly optimistic solution since the redundancy in the network (degrees of freedom) is artificially increased. This may produce an adjustment that appears statistically superior. Station and relative errors can be affected if trivial baselines are included in the network adjustment.

Processing of a baseline should result in a fixed solution. The statistical quality of all baselines should be reviewed. Table 8-2 indicates generally acceptable values.

RMS < 0.04	Ratio > 1.5	Reference Variance < 15

Table 8-2
Generally Acceptable Baseline Results

The processed baselines (SSF/SST files) are saved by default to the GPSurvey™ project or the TGO™ project.

8.5.3 NETWORK ADJUSTMENT

A Network adjustment is typically a sequential process. First, a minimally constrained horizontal adjustment is run. After the satisfactory completion of the fully constrained horizontal adjustment, a fully constrained adjustment is completed. All adjustments will be preformed relative to NAD83(1992) or NAD83(1999)/NAD83(CORS) and not WGS84.

If TGO™ is used, the Trimble supplied adjustment style "95% Confidence Limits" will be used. **A new adjustment style will not be created**, but options within this style may be modified such as standard errors associated with antenna heights and the use of a correlated geoid model.

8.5.3.1 General Guidelines

Prior to an adjustment, loop closures should be analyzed and redundant baseline vectors compared. Refer to Table 8-3. Suspect baselines should not be disabled or deleted prior to the minimally constrained or a free adjustment. Excessive loop closures may be improved by reprocessing baselines having low ratios or high reference variances using a higher mask or by elimination of selected satellites. An obviously erroneous baseline may be disabled or deleted at this time. It is advisable to disable a baseline versus deletion of that baseline.

Maximum Loop Closure (3D)	Not greater than 20ppm
Maximum Loop Closure (X, Y,Z)	Not greater than 0.15 ft (0.05m)
Redundant Baseline Comparison (3D)	Not greater than 10ppm
Redundant Baseline Comparison (X, Y, & Z)	Not greater than 0.10 ft (0.03m)
Maximum Residual (in any component) in a Weighted Least Squares Adjustment.	Not greater than 0.13 ft (0.04m)

Table 8-3
Suggested Maximum Loop Closures and Redundant Baselines Comparisons

A minimum of three baselines is required to produce a loop. Each loop must contain baselines associated with a minimum of two different sessions. A loop closure consisting of baselines from a single session is meaningless. A receiver may not remain at a point during different sessions, unless the height of the antenna is changed between sessions.

- LC = $\sqrt{\Delta X^2 + \Delta Y^2 + \Delta Z^2}$ where LC = Loop Closure
- ppm = (LC $\div$ TLD) x 1,000,000 where TLD = Total Loop Distance

- MLC = (ppm x Total Loop Distance) ÷ 1,000,000 where MLC = Maximum Loop Closure

The maximum loop closure and redundant baseline comparisons should meet the smaller value of either the ppm or the closure in X, Y, Z. Generally the closure in X,Y,Z will be controlling. For example, assume a Loop Closure was equal to 0.06m and the Total Loop Distance = 4000m. 0.06m exceeds the maximum loop closure of 0.05m. The Maximum Loop Closure based on 20ppm would be 0.08m. Therefore the maximum closure for this loop would be 0.05m and not 0.08m.

The above are general guidelines with the exception of maximum residuals. A residual greater than 0.13 ft (0.04m) is a very good indication that excessive random errors, or systematic errors, and or mistakes are present in the observations.

8.5.3.2 Minimally Constrained or a Free Adjustment

After excessive loop closures and differences in redundant vectors have been resolved, a minimally constrained adjustment can be run. The purpose of the minimally constrained adjustment is to reveal any problems associated with the internal network. A minimally constrained adjustment will hold the geodetic coordinates, (latitude and longitude) of one of the primary control marks (HARN) fixed. The ellipsoid height may be constrained at the same point or the ellipsoid height will be determined from the inner constraints. A free adjustment (no mark constrained) is also acceptable. The advantage of a minimally constrained adjustment as compared to a free adjustment is that the coordinates generated for the unconstrained primary project control marks (HARN) may be compared with the published values. An unsatisfactory comparison will create problems during the fully constrained adjustment and should be resolved prior to the fully constrained horizontal adjustment.

General Guidelines The initial adjustment should include estimated errors associated with the centering of the antenna and the height of the antenna. Centering errors should be estimated at or near 0.005 ft (0.0015m) and height errors estimated at or near 0.007 ft (0.0020m). A network adjustment should be preformed with the scalar set at the default (1.00) and the above estimated errors. This adjustment will be performed at the 95% confidence level. The network statistics and residuals should be reviewed. The Net Work Reference Factor (NWRF) should be less than 6.00. A NWRF greater than 6.00 may indicate problems in the network and additional analysis or observations may be required. A scalar may be applied and the network readjusted using an alternate scalar until the NWRF approaches 1.00. At this time, there should not be any significant outliers or residuals. If significant outliers or excessive residuals are present, additional analysis and/or observations may be required. If a scalar was used, it should be set (user defined) at this time.

8.5.3.3 Fully Constrained Adjustment

After the minimally constrained adjustment has been successfully completed, a fully constrained adjustment will be run. The purpose of the fully constrained adjustment is to reveal any problems with the existing control (HARN, previous Department control marks, OPUS and/or CORS) and to provide coordinate information of all marks included in the network. Geodetic coordinates, not the corresponding state plane coordinates, of the existing control will be constrained. All final project coordinates will be relative to NAD83(1992) or NAD83(1999)/NAD83(CORS).

General Guidelines Typically the fully constrained adjustment is a three step process. Initially all known geodetic coordinates (latitude and longitude) are constrained. The ellipsoid heights may be constrained at each known mark or the ellipsoid heights may be determined from the adjustment. The NWRF and statistics should be reviewed. The NWRF may increase from the minimally constrained adjustment, but there should not be a significant change in residuals or outliers. The scalar as determined in the minimally constrained adjustment should not be increased or decreased.

After the satisfactory completion of the fully constrained horizontal adjustment, the appropriate geoid model is loaded and the elevation of one vertical mark is constrained. All ellipsoid heights should be freed for the vertical orthometric adjustment. The network adjustment is run. At this time the GPS derived orthometric heights (elevations) should be compared to the published or known elevations of all other known marks included in the network. This comparison should reveal vertical problems associated with any of the remaining vertical marks.

Depending upon the vertical results, additional vertical control marks may be constrained and a final adjustment run. The result of this adjustment is then used to create the first of two required coordinate lists. Refer to deliverables below.

8.5.4 DELIVERABLES

The land surveyor in responsible charge of a GPS control survey associated with a Department developed project will finalize all information associated with the survey. The land surveyor or designee will upload all required files to the appropriate project folder in the Document Management System (DMS).

Consultant GPS control surveys require all deliverables to be submitted on a compact disk (CD) to Consultant Design. Files names will conform to DMS naming conventions. Submittal of data on a floppy disk(s) is not acceptable. **Original** field notes will also be submitted to Consultant Design. All GPS control data will be forwarded to Photogrammetry/Survey for finalization and uploading of the data to the appropriated folder in the DMS.

GPS control surveys done by or for the Department require the following deliverables:

- Original obstruction diagrams, field notes and/or original observation logs
- Control mark abstract
- Raw files (RINEX-navigation, observation, and RNX files and/or Trimble™ DAT files)
- Baseline results that include from, to, baseline length, solution type, ratio, reference variance, and if available the (rms)
- Final observation schedule
- Solution files (SSF and/or SST)
- Minimally constrained network(s)
- Fully constrained network(s)
- GPSurvey™ archive or TGO™ WinZip™ file
- NGS data sheets
- Two coordinate lists-NAD83(1992), or NAD83(1999)/NAD83(CORS)
- OPUS solution reports
- Terrestrial observations
- Control diagram
- Project combination scale factor(s)
- Surveyor’s report

8.5.4.1 Clarification of Certain Deliverables

Original Field Notes Original field notes are the notes as made during the observation. **Scans or copies** of the original field notes are not acceptable.

Control Mark Abstract The control abstract will be an American Standard Code for Information Interchange (ASCII) text file. This abstract will include all marks occupied by GPS and/or conventional instruments. The abstract will be sorted alphanumeric and in the same order as the coordinate lists. Refer to Form 5-1.

Final Observation Schedule Refer to Form 8-3.

Minimally Constrained Network Included within the GPS project files.

Fully Constrained Network This represents the final GPS adjustment. Included within the GPS project files.

GPS Project Files Networks adjusted using GPSurvey™ will be archived using the "backup everything" option. This will create a file name *.A1. Networks adjusted using TGO™ will be zipped to a file using WinZip™. The A1 file or the zip file will include all folders and files within all folders associated with either the GPSurvey™ or the TGO™ project.

NGS Data Sheets NGS data sheets for all horizontal and vertical marks that were occupied during the survey. This includes data sheets for vertical marks that were not directly occupied, but required the setting of a TBM.

Coordinate Lists Two coordinate lists will be submitted in a **space delimited ASCII text format** and not in Word, Word Perfect, or a spread sheet format.

The first coordinate list will be based on the results of the final fully constrained network adjustment and will be sorted alphanumeric based on the mark name. Refer to Form 8-4. This listing will indicate the mark name, latitude, longitude, ellipsoid height, state plane north, state plane east, elevation (GPS derived), CSF, and convergence of all marks included in the adjustment. The listing will also indicate which marks were constrained in the final adjustment (horizontal and vertical) and the source, the horizontal datum (including the adjustment tag), the vertical datum, and the units associated with the state plane coordinates. Latitude and longitude will be shown to the nearest 0.00001", ellipsoid height to the nearest 0.01 ft (0.001m), north and east to the nearest 0.001 ft (0.0001m), elevations to the nearest 0.01 ft (0.001m), mark CSF to the nearest 0.00000001, and the convergence to the nearest second.

The second coordinate list will contain all final state plane coordinate information associated with all marks and will be sorted alphanumeric. Refer to Form 8-5. Information included in this file will be mark name, state plane north, state plane east, and elevation (final elevations for the project control). This listing will also indicate the horizontal datum including the adjustment tag and the vertical datum. Horizontal coordinates will be shown to the nearest 0.001 ft (0.0001m) and elevations to the nearest 0.01 ft (0.001m).

The number of marks and the elevations on Form 8-5, may not be identical to the corresponding information in the previous coordinate listing (Form 8-4). For example, additional marks may have been located using only terrestrial observations and these observations were not incorporated into the network adjustment. This coordinate file (Form 8-5) will contain any additional marks not included in the first coordinate listing.

Another example. All known elevations were not constrained in the final adjustment. If the difference in elevation between the GPS derived and the published elevation is not significant, the GPS derived elevations (as shown in the first coordinate list) will be replaced with the published NGS elevations. If the difference is significant, the mark in question will have two elevations. One noted as a project elevation the other noted as a published elevation.

Another example. Differential levels were established through the control marks and this information was not incorporated into the fully constrained adjustment. In this case either the GPS derived elevations will be shown or the GPS elevations will be replaced with the differential elevations. In all cases, the information in this file will be considered to be the final coordinate values associated with the GPS control survey.

OPUS Solution Reports ASCII format and includes extended output options for all OPUS solutions obtained during the project.

Terrestrial Observations Terrestrial observations may include conventional horizontal control survey information to marks that could not be occupied by GPS, and/or differential levels. All conventional survey will adhere to the control closure specifications (horizontal and vertical) as outlined in Chapter 4. Required deliverables, beside the original field notes, will include all raw data collector files, all adjustment reports, and a final coordinate list. This coordinate list will consist of only marks that were used in the conventional surveys.

Control Diagram Consultant projects only. This file will be a MicroStation DGN file. The current seed file and the current MicroStation version will be used. This DGN file will not contain any reference files. This DGN file will be full scale with no rotation.

Project Combination Scale Factor(s) One or more CSF(s) may be required. The number of CSF(s) associated with a project is a function of the total change in latitude and elevation. Marks outside the project limits should not be included in the development of the CSF(s).

An analysis of CSF associated with each mark will determine the number of CSF(s) required. As a guideline, small projects such as bridge relocations usually require only one CSF. Larger projects that extend east/west may require one. Larger projects that extend north/south may require two or more CSF. The mean CSF can be determined using the average latitude and elevation of the project or it can be based on a mean of the individual CSF of the marks in the immediate vicinity of the project. The determining factor is that the reciprocal of the differences in the CSF must be greater than 1:50000. As an example, assume the mean CSF is determined to be 0.99948322. The smallest CSF = 0.99945344. Therefore 1/(0.99948322 – 0.99945344) = 33580. Conclusion - one CSF is not sufficient. Therefore there would be at least two CSF(s) associated with this project.

Surveyor's Report The land surveyor in responsible charge of the GPS control survey will provide what is also known as a read me file. This will be an ASCII text file that includes but not limited to the following:

- A brief explanation of the contents of all files included on the CD.
- A listing of the primary control marks (HARN, previous Department control, bench marks, etc) constrained in the final adjustment.
- CSF(s) associated with the project and how developed.
- Coordinate units (metric, U.S. survey feet, or international feet).
- Horizontal datum (including the adjustment tag) and vertical datum.
- Product name and version of software used for the adjustment.
- A statement that all survey conforms to MDT guidelines.

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8.6 GPS HARN DENSIFICATION

The Department may require additional marks to be observed and incorporated into NGS's National Spatial Reference System (NSRS). These surveys must meet all NGS requirements such as equipment, occupation times, observation logs, and connection to existing control. All Department HARN densification surveys will be coordinated through Photogrammetry & Survey and the state Geodetic Advisor.

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8.7 CADASTRAL SURVEYS AND CONTROL DENSIFICATION SURVEYS

In addition to the establishment of project control, the Department also uses survey grade GPS for cadastral surveys (land surveys) and minor densification of existing GPS control surveys. Cadastral surveys and densification of existing control surveys may be accomplished using kinematic methods. However, cadastral surveys and minor densification of an existing GPS control survey are not restricted to only kinematic methods. Cadastral and densification surveys may utilize a combination of static, fast static (rapid static), conventional surveys (total stations and field notes or total stations and data collector), as well as post processed kinematic and real time kinematic .

Conventional surveys methods may be more feasible for minor densification of an existing project control survey than GPS. All conventional surveys will be conducted using the control survey method, secondary traverse method, or radial survey method. Tolerances and procedures for conventional surveys may be found in Chapters 4, 5, and 6.

Kinematic surveys are typically radial in nature and may be classified as Post Processed Kinematic (PPK) or Real Time Kinematic (RTK). Each method has advantage and disadvantage. The primary advantage of PPK is a radio link is not required. The major disadvantage of PPK is the data requires processing to review the results.

All coordinates associated with a cadastral survey or a minor densification of the existing GPS control survey will be relative to the coordinate system established during the control survey.

8.7.1 RECEIVERS AND ANTENNAS

Survey grade receivers are required. Single or dual frequency receivers may be used. A minimum of two receivers is required. The base receiver (reference receiver) should be placed on a known control mark that was established during the original GPS control survey. The antenna at the base receiver should be attached to a tripod or a fixed height tripod. The antenna will utilize a ground plane.

The configuration of the roving receiver depends upon the type of survey. A tripod and ground plane is not required for cadastral surveys. A control densification survey requires a tripod and ground plane.

8.7.2 RECORDING INTERVALS AND OCCUPATION TIMES

PPK and fast static surveys require post processing of data. A recording interval of 5 to 15 seconds is typical of PPK and occupation times may vary from 30 seconds to several minutes. Fast static data is usually collected at 15 seconds interval and a minimum occupation time of nine (9) minutes.

RTK surveys result in a coordinate of the occupied mark at the time of observation. Post processing of the data is not required. The recording interval is generally set at a 1 second interval. Occupation times vary from a few seconds to several minutes.

8.7.3 FIELD NOTES

Field notes are required. Kinematic (PPK or RTK) field notes may be recorded in a standard field notebook or recorded on special forms. Refer to Form 8-6 and 8-7. Form 8-7 is a continuation of Form 8-6 and should not be used in lieu of Form 8-6. The page number information (bottom right hand corner) must be completed for both forms. In some cases Forms 8-6 and 8-7 may provide enough space to adequately describe the monument that was found or set. In other cases supplemental field notes should be recorded in a standard field notebook. Fast static field notes may be recorded in a standard field notebook or on special forms. Refer to Form 8-2. See Appendix A, Figure A-28 for an example of standard field notes.

The above forms or supplemental field notes must contain enough information to prepare an abstract for the additional control marks, or contain enough information for the preparation of a corner recordation and/or a certificate of survey(s).

8.7.4 ABSTRACTS AND MARK ID (POINT NUMBER)

8.7.4.1 Cadastral Surveys

Cadastral surveys generally do not require a separate file containing the descriptions associated with the found or set monuments (property corners and property controlling corners). This information is included in the final coordinate list and on the certificate of survey. The point number in all files will correspond with the naming convention in Chapter 6.

8.7.4.2 Minor Densifications of an Existing GPS Control Survey

At the time of the original control survey (conventional or GPS), control marks should have been placed at all locations necessary to complete the preliminary surveys. Additional control marks should not be required. However, as the preliminary surveys are completed, additional control marks may be necessary. Refer to Figure 6-5. The new control marks will consist of a rebar (5/8-inch x **30-inch** (16mm x 762mm), an appropriately stamped 2 inch (51mm) aluminum cap, and a witness post. An abstract is required.

Densification survey requires a separate abstract text file. This file contains a complete and accurate abstract for each new control mark. Additional newly established control mark names will conform to the naming convention in Chapter 5. Refer to Form 5-1.

8.7.5 SOFTWARE

The initial GPS control survey requires a fully constrained adjustment using either GPSurvey™ or the latest version of TGO™. Cadastral surveys and minor control densification surveys may also use this software. In addition, software such as Star*Net™, Star*Net-Pro™, GEOPAK Survey™, or other approved software packages may be used for cadastral surveys and minor densification surveys.

8.7.6 DATA COLLECTION

Kinematic surveys require an initialization. This initialization can be accomplished “on the fly” or using a “known point”. RTK requires an initialization prior to the collection of any data. PPK does not, since the collected data will be processed prior to and after initialization has been obtained. It is advisable to initialize PPK surveys prior to the collection of data or immediately after lock is lost.

If initialization is obtained “on the fly” using RTK, a check observation should be made prior to the occupation of any new mark. It is recommended a final check observation be made prior to ending the kinematic survey, or at any time a new initialization is obtained. The purpose of this check is to determine if:

- the base receiver is at the correct mark
- the mark at the base has not been disturbed
- the radio and associated hardware is connected correctly

- correct coordinates of the base have been downloaded to the data collector
- excessive errors do not exist in the antenna heights
- a correct initialization was obtained

During a RTK session, the data collector may display information such as RMS, solution type, horizontal precision, and vertical precision. Suggested precisions for acceptance are:

- rms is less than or equal to 50
- solution type-fixed
- horizontal precision less than or equal to 0.04 ft (0.012m)
- vertical precision less than or equal to 0.07 ft (0.021m)

The above values do not indicate accuracy of the observation. They indicate how well the individual baseline solutions within each occupation are agreeing with each other.

All marks occupied by the rover must utilize a minimum of two independent or at least two quasi-independent observations. Quasi-independent observations are obtained by two closely observed occupations of the same mark.

Caution As noted earlier in this chapter, if two fast static bases are being utilized and a rover occupies a monument the generation of a baseline from each base to the rover position does not constitute two independent observations. This also applies to kinematic surveys (RTK and/or PPK). If kinematic is used in conjunction with two bases, the coordinate of the rover position as determined from each base does not constitute two independent observations. If the coordinate of a mark occupied by the rover is obtained using RTK from one base, and a post processed observation (fast static or a PPK) is obtained from the other base, this procedure does not constitute two independent observations. Independent observations may only be obtained if there is a significant time period between the two occupations of the mark by the roving receiver.

The point number assigned to the observed monument in all data files (conventional data collector and/or GPS) will correspond to the point numbering format associated with control marks or the point numbering format associated with property and property controlling corners. Refer to Chapter 5 and 6.

8.7.6.1 Observation Scenarios

In the case of radial surveys, all monuments associated with a cadastral or a control densification surveys will result in two coordinate values for all observed marks. The two coordinate values may be determined using conventional instruments (total stations and field notes or total station and data collector), GPS (post processed and/or RTK), or a combination of conventional instruments and GPS.

Conventional Instruments Only-Specifications Refer to Chapter 5 and 6 for procedures. Refer to Chapter 4 for traverse closures and maximum radial differences associated with radial surveys.

Minimum GPS Only-Specifications (Quasi-Independent) All property corners and property controlling corners associated with cadastral surveys will result in a minimum of :

- two post processed observations from a single base receiver or
- two RTK solutions from a single base receiver or
- one RTK solution and one post processed observation from a single base receiver

General procedure:

- obtain either an RTK solution **or** an observation (post processed)
- lose initialization, obtain an “on the fly” initialization, and reoccupy the mark **or** obtain another observation (post processed) to the mark

If RTK solutions are used exclusively, a check observation to a known mark should be made prior to the collection of data. The quasi-independent RTK solutions will be separated by a minimum of 15 minutes. Refer to Figure 8-9(A)

Caution The above procedures are not to be used for densification of existing control surveys and **should not be standard procedure** for cadastral surveys. Densification surveys will conform to the recommended GPS only specifications or the recommended combination of conventional instruments and GPS as explained below.

Recommended GPS Only-Specifications All property corners and property controlling corners associated with cadastral surveys or with the densification of existing GPS control surveys will result in at least two independent observations to all marks occupied by the roving receiver. Independent observations can be obtained by a post processed observation from two different base locations, or a RTK solution from two different base locations, or a post processed observation from one base and a RTK solution from a different base location. The post processed observations or the RTK solutions should be separated by a minimum of two hours. This can be accomplished by occupying several marks in the morning with the base receiver at a control mark. At the completion of the occupations, the base receiver is moved to a different control mark and the same marks are once again occupied with the roving receiver in the afternoon. Independent observations can also be accomplished with a base receiver at a control mark and the roving receiver occupies selected marks during day one. On day two, the base receiver is moved to a different control mark and the marks observed during day one are once again occupied. It may be advisable to occupy the marks in the reverse order of the day one occupations. If a mark was occupied on day one at 9:30AM this mark should not be occupied on day two from 7:30AM to 11:30AM. Refer to Figure 8-9(B). Cadastral surveys are usually radial. Densification of project control may also be radial, or consist of loops, or a combination of radial and loop methods. See Figures 8-6 and 8-7.

Recommended Combination of Conventional Instruments and GPS The monuments associated with the cadastral or densification survey may be observed using conventional instruments, GPS, or a combination of conventional instruments and GPS. There are no constraints as to time intervals between the conventional observation and the GPS observation. The GPS observation may be either a RTK solution or a post processed observation. The conventional survey may use a total station and field notes or a total station and a data collector.

During the cadastral survey, a monument may not be able to be occupied by GPS due to obstructions (dense trees, fence lines, or fence post). Conventional survey methods may be used to make a tie to the obstructed point. Refer to Figure 6-1 to Figure 6-4. In lieu of conventional survey methods, a combination of conventional survey methods and GPS may be used. Refer to Figure 8-10. GPS is used to establish two random points near the obstructed mark. Each random point is occupied with a total station and radial ties are made to the obstructed mark. The distance between the random points should be greater than the distance to the obstructed mark.

An alternative, to the establishment of two random points, is to occupy the obstructed monument with a total station and place two points on line. Refer to Figure 8-11. The distance to the nearest point on line from the obstructed monument should be as short as possible. The distances between the two points on line should be significantly greater than the distance between the obstructed monument and the nearest point on line. The distances are then measured with the total station and the two points on line are observed using GPS techniques. Once coordinates of the two points on line have been determine, the coordinate of the obstructed point may be determined.

8.7.7 COORDINATES-GENERAL

The radial survey specifications outlined above results in two coordinate values for each observed mark. In the case of cadastral surveys, the radial difference (horizontal inverse) between the two coordinate values must be equal to or less than 0.25 ft (0.076m). The radial difference between the two coordinates values associated with a densification project must be equal to or less than 0.10 ft (0.03m). Final coordinates associated with radial survey methods may be determined by:

- merging of duplicate points (a TGO™ option)
- least squares
- or a mean of the individual coordinates.

If a conventional traverse(s) is used, closures (angular and ratio) must adhere to the specifications associated with conventional surveys. Final coordinates associated with a conventional traverse(s), for the purpose of control densification or a traverse between azimuth pairs, will be determined using either a compass or least squares adjustment. Elevations will be determined using differential levels. Elevations are not required for secondary traverses associated with cadastral surveys.

8.7.7.1 Merging of Duplicate Points or Least Squares

The merging of duplicate points and least squares adjustment requires the observed mark to have the same name during all occupations. For example on day one, a rover receiver occupied property controlling corners identified as 205-209. On day two, the rover occupied these same property controlling corners. The mark names in both GPS files would be 205-209. The same procedure would apply if conventional survey methods were used.

If Trimble™ receivers are used, QC1 and QC2 data may be collected. RTK data containing this information can be incorporated into a network adjustment.

8.7.7.2 Mean Coordinates

In order to mean coordinate values it may be necessary to assign unique names to the occupied marks. For example on day one, the rover occupied three property corners and they were identified as A433, A434, and A435. On day two, the rover occupied these same property corners, but they were now identified as B433, B434, and B435. The coordinates of the marks are averaged, and the final point number associated with the mean coordinate value will be 433, 434, and 435, and not C433, C434, or any other variation.

8.7.7.3 Set Monuments-Cadastral Surveys

Cadastral surveys consist of ties to found monuments as well as surveys required to set monuments at a calculated position. The final coordinates of found monuments will be based on the previous sections and these coordinates will be maintained in the final cadastral coordinate list. The coordinates of set monuments (calculated) will also be maintained in the final cadastral coordinate list. After a monument has been set, the coordinate position of that monument will be verified. The radial differences between the calculated position and the set position must be less than or equal to 0.25 ft (0.076m). If the radial difference (based upon a horizontal inverse) is greater than 0.25 ft (0.076m), additional survey will be necessary to determine if the monument was set correctly. Several methods may be used to verify if a monument was set correctly. All methods consist of obtaining any necessary observations after the monument is set to compute the coordinates of the monument.

Examples would be:

- Conventional instruments
  - the monument is set
  - a different backsight is obtained
  - angle and distance are measured to the set monument
  - coordinates are computed for the set monument and
  - coordinates are compared to the calculated position
- GPS
  - the monument is set with the base receiver on a control mark
  - the base is moved

  - a PPK or a fast static observation, or a RTK solution is obtained to the set monument
  - coordinates are computed for the set monument and
  - coordinates are compared to the calculated position

- Conventional and GPS
  - the monument is set using either conventional instruments or GPS
  - observations are obtained to the set monument using the opposing method (i.e. if set conventionally, check with GPS or vice versa)
  - coordinates are computed for the set monument and
  - coordinates are compared to the calculated position

8.7.8 DELIVERABLES

8.7.8.1 Control Densification

Conventional densification surveys require the deliverables outlined in Chapter 5. Electronic survey information associated with a Consultant Design survey will be provided to the Consultant Design Bureau on a compact disk along with all **original** field notes. All file names will conform to DMS file naming conventions. Consultant Design will forward this information to Photogrammetry & Survey Section for finalization and uploading to DMS.

All survey information including data collector files (conventional and GPS), computations, coordinates, abstracts, field notes, and differential levels will be provided. A land surveyor will finalize all control densification surveys. The land surveyor or designee will then upload all relevant files to the appropriate project folder in the DMS. In addition, a memo should be sent to the lead bureau indicating the densification of the control survey has been completed.

The following files are required and will be uploaded to the appropriate folder using DMS:

- all raw data collector files
- final ASCII coordinate file (format P N E Z) of all additional control marks

- control abstract of all additional control marks
- all adjustment reports (TGO™ project files, Star*Net™ files, Star*Net-Pro™ files, etc)
- updated control diagram-consultant projects only
- read me file

8.7.8.2 Revised Control Diagram

After the conventional control survey (Chapter 5) or the GPS control survey has been completed and the required deliverables have been placed in the appropriated project folder, the initial control diagram can be completed. The control diagram for Department projects are typically created by Road Design or Traffic. The control diagram for consultant projects are completed by the consultant. This initial control diagram will not include any new control marks associated with subsequent control densification surveys. Therefore, the initial control diagram should be updated to reflect all control marks associated with the project. The control diagram will **not** include the random points that were established in conjunction with topographic surveys.

8.7.8.3 Cadastral Surveys

Electronic survey information associated with a Consultant Design project will be provided to the Consultant Design Bureau on a compact disk along with all **original** field notes. All file names will conform to DMS file naming conventions. Consultant Design will forward this information to Photogrammetry & Survey Section for finalization and uploading to DMS. Cadastral surveys performed by Department personnel will also be uploaded using DMS.

All survey information including data collector files (conventional and GPS), computations, coordinates, abstracts, field notes, and differential levels will be provided. A land surveyor will finalize all cadastral surveys. The land surveyor or designee will then upload all relevant files to the appropriate project folder in the DMS. In addition, a memo should be sent to right-of-way indicating the cadastral survey has been completed.

Conventional cadastral surveys require the deliverables outlined in Chapter 6. Cadastral surveys that utilize GPS requires the following **additional** files and they will be uploaded to the appropriate folder using DMS:

- all raw data collector files (horizontal and vertical)
- all adjustment reports (TGO™ project files, Star Net™ files, Star*Net Pro™ files, etc
- a final ASCII coordinate file, listing all points observed and calculated (format P N E D)
- a final ASCII coordinate file, listing that includes only information associated with the recorded certificate of survey (format P N E D)
- as recorded certificate(s) of survey-must be in MicroStation format
- as recorded corner recordations-optional
- a read me file

In all cases, the Right-of-Way Bureau or District Right-of-Way Design will be provided paper copies of all as recorded certificates of survey and corner recordations.

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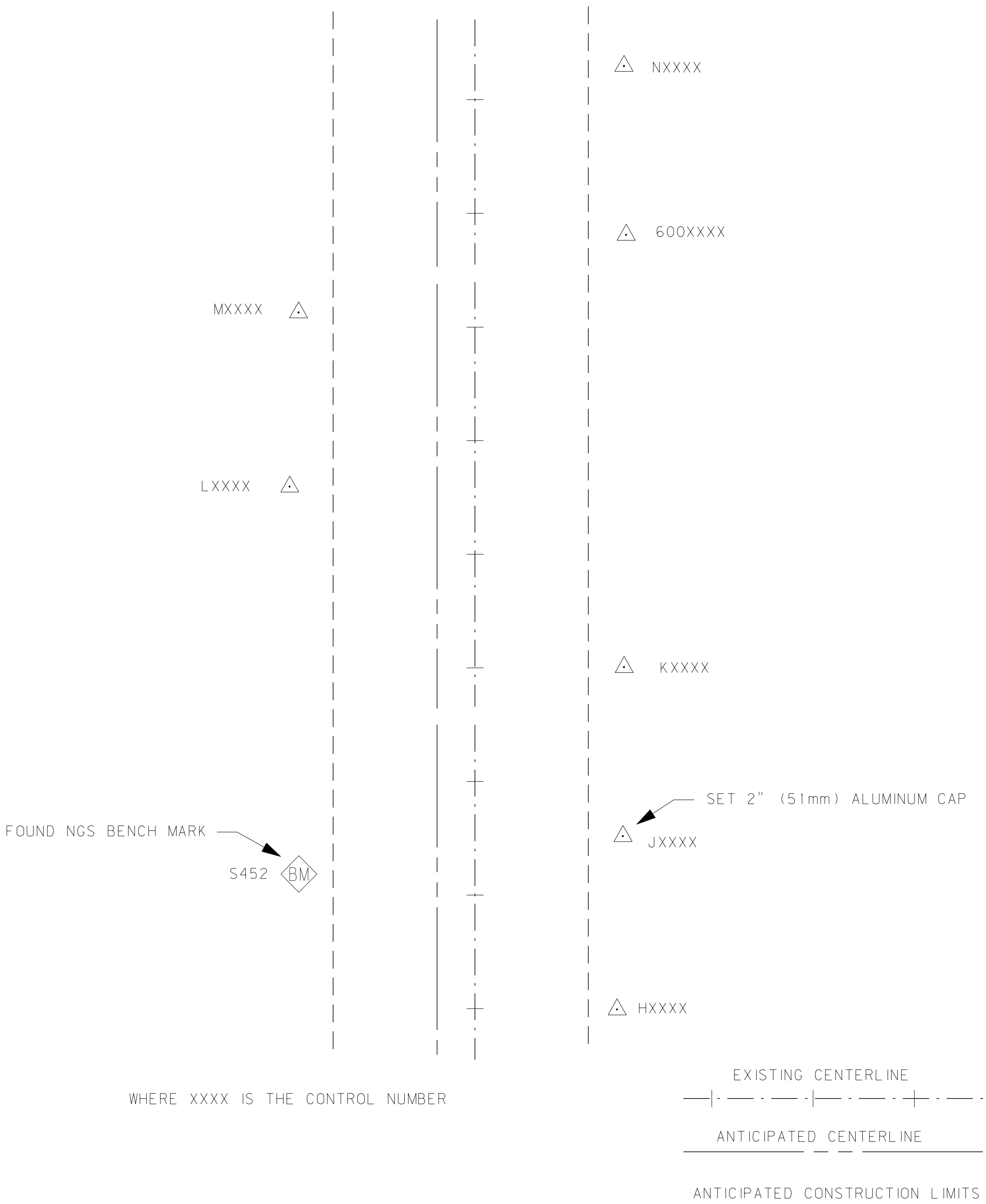


Figure 8-1
Typical GPS Project Control Mark Locations

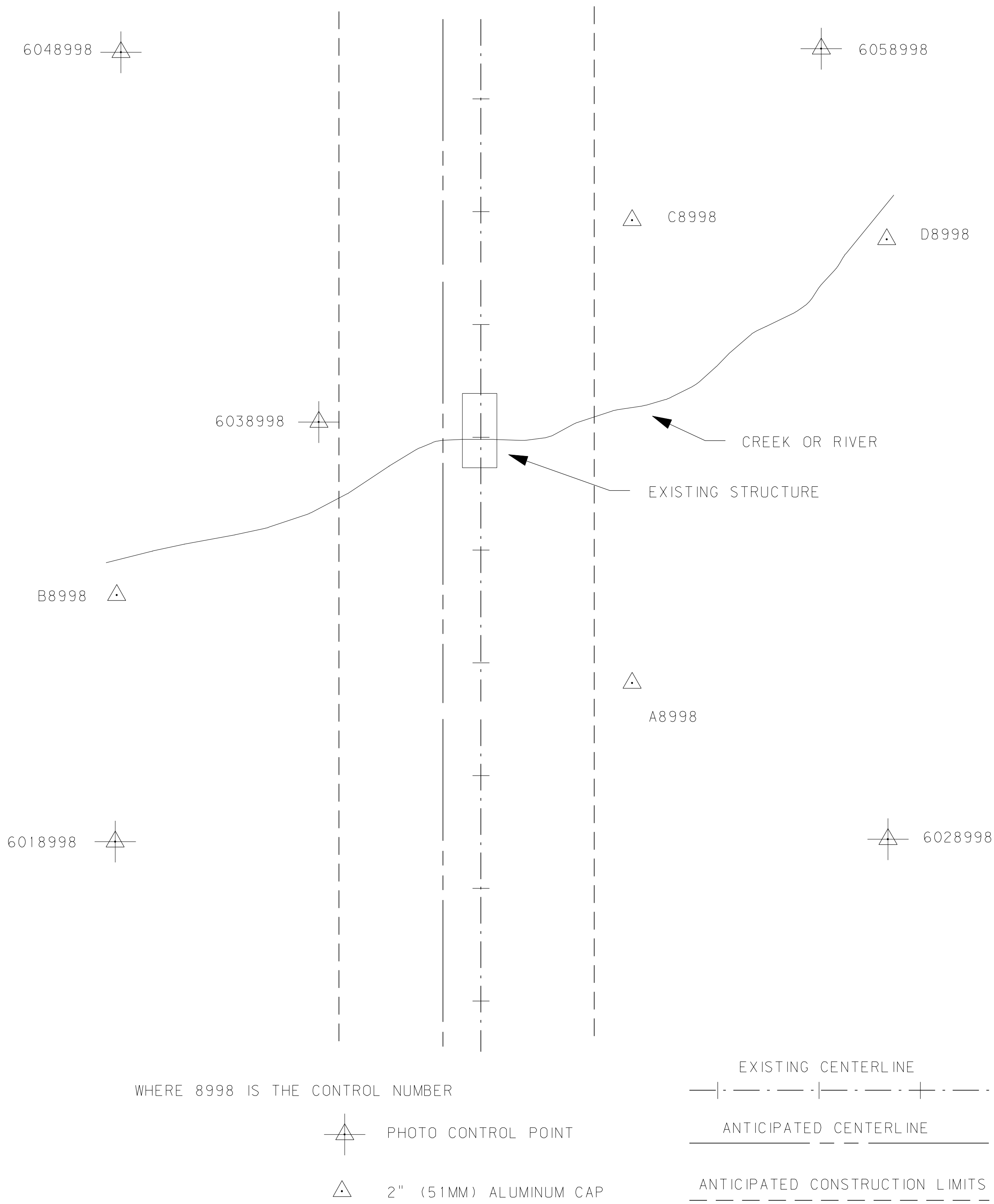


Figure 8-2
Typical GPS Project Control Mark Locations

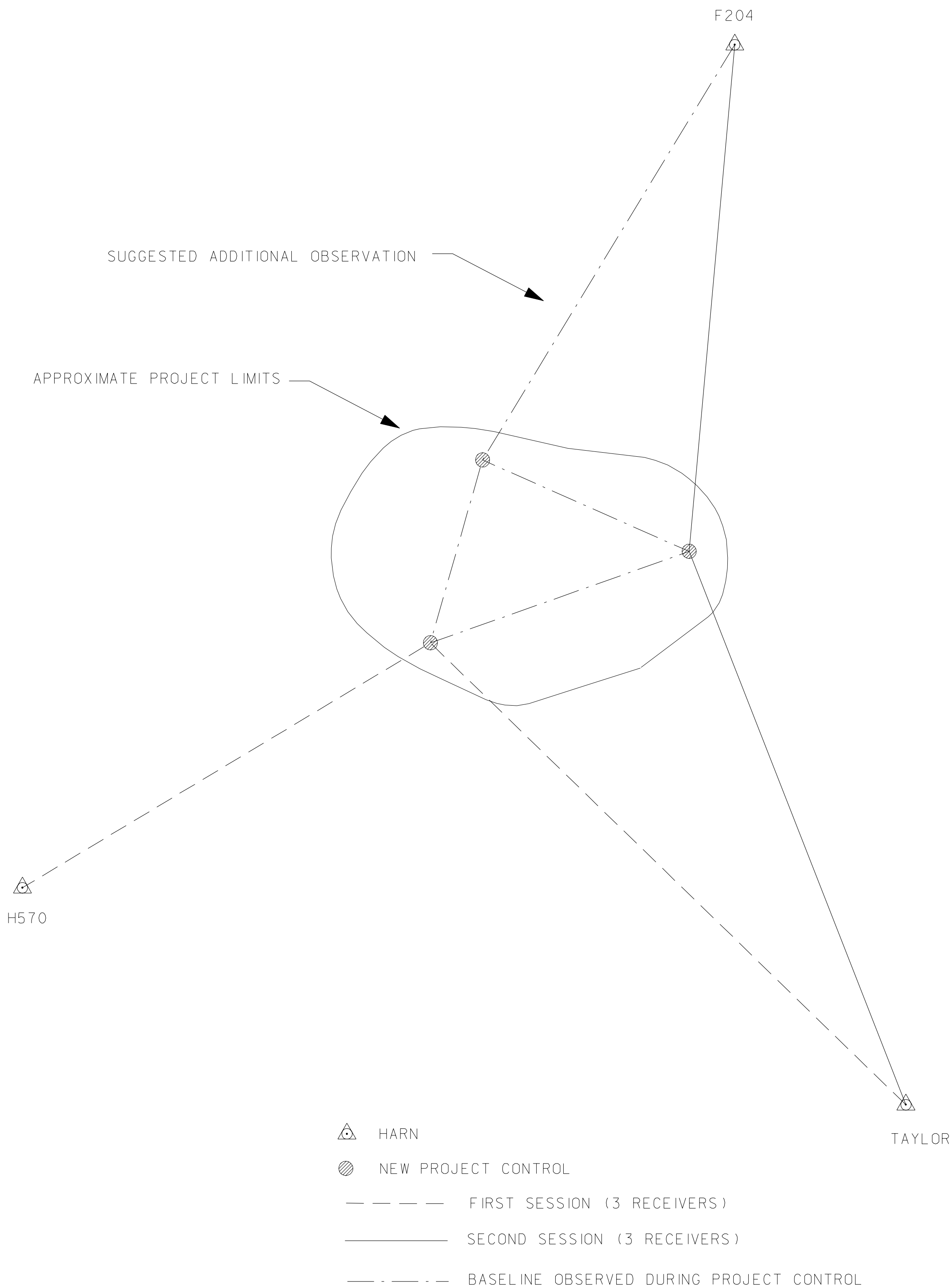


Figure 8-3
Project Connection to HARN Marks

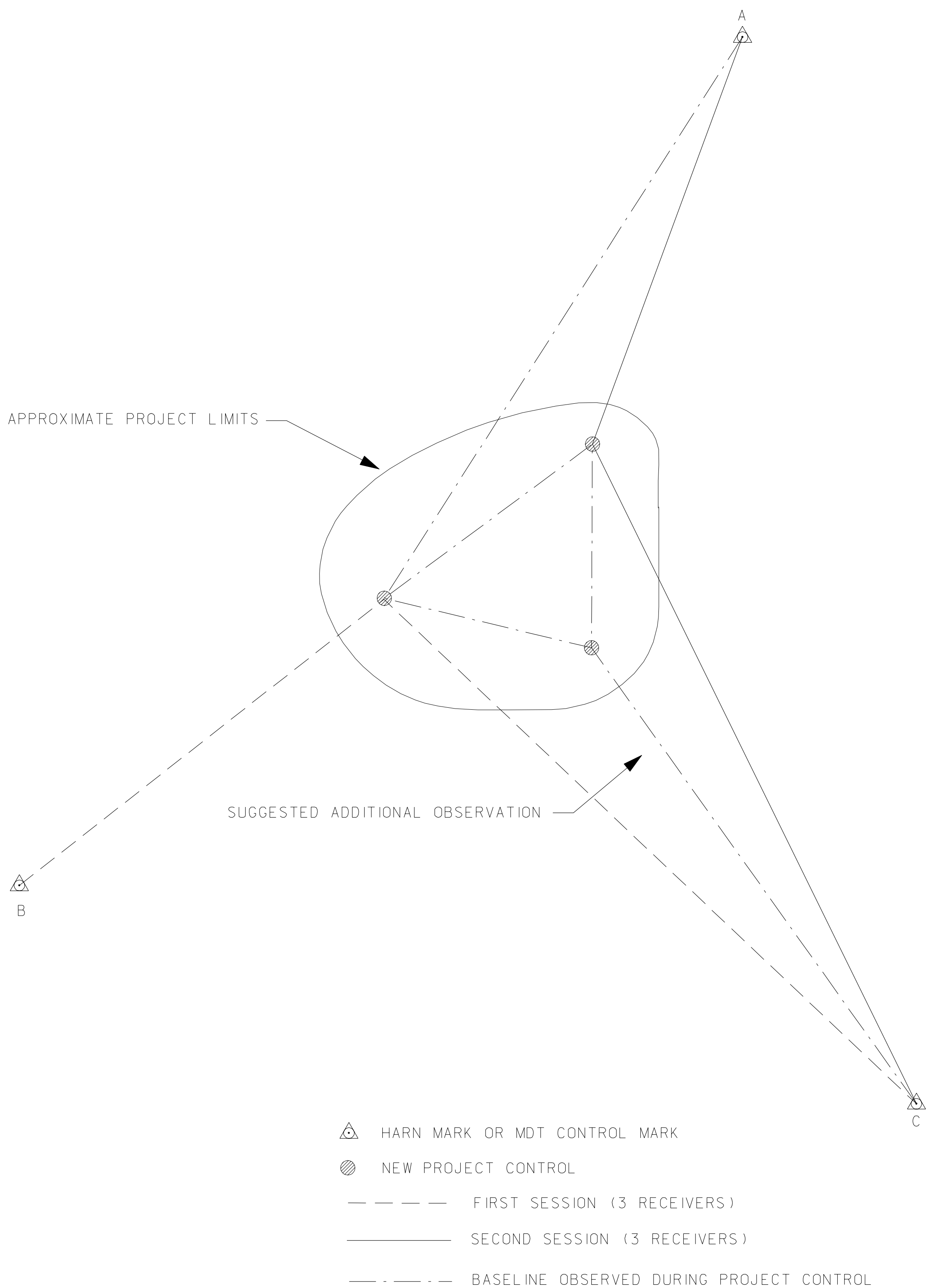


Figure 8-4
Project Connection to HARN and Previous MDT Project Marks

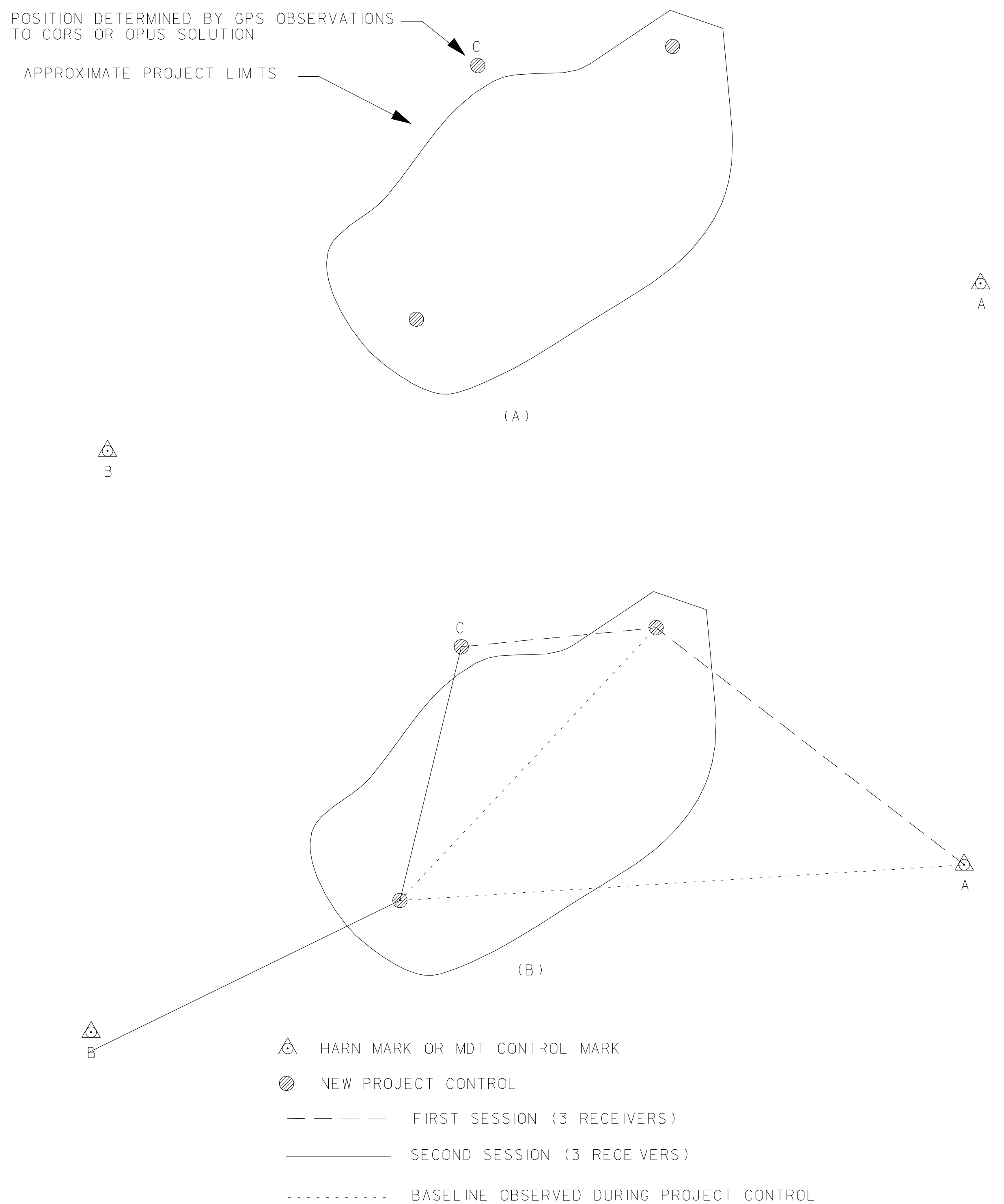


Figure 8-5
Project Connection to CORS, HARN, Previous MDT Project Marks and/or OPUS

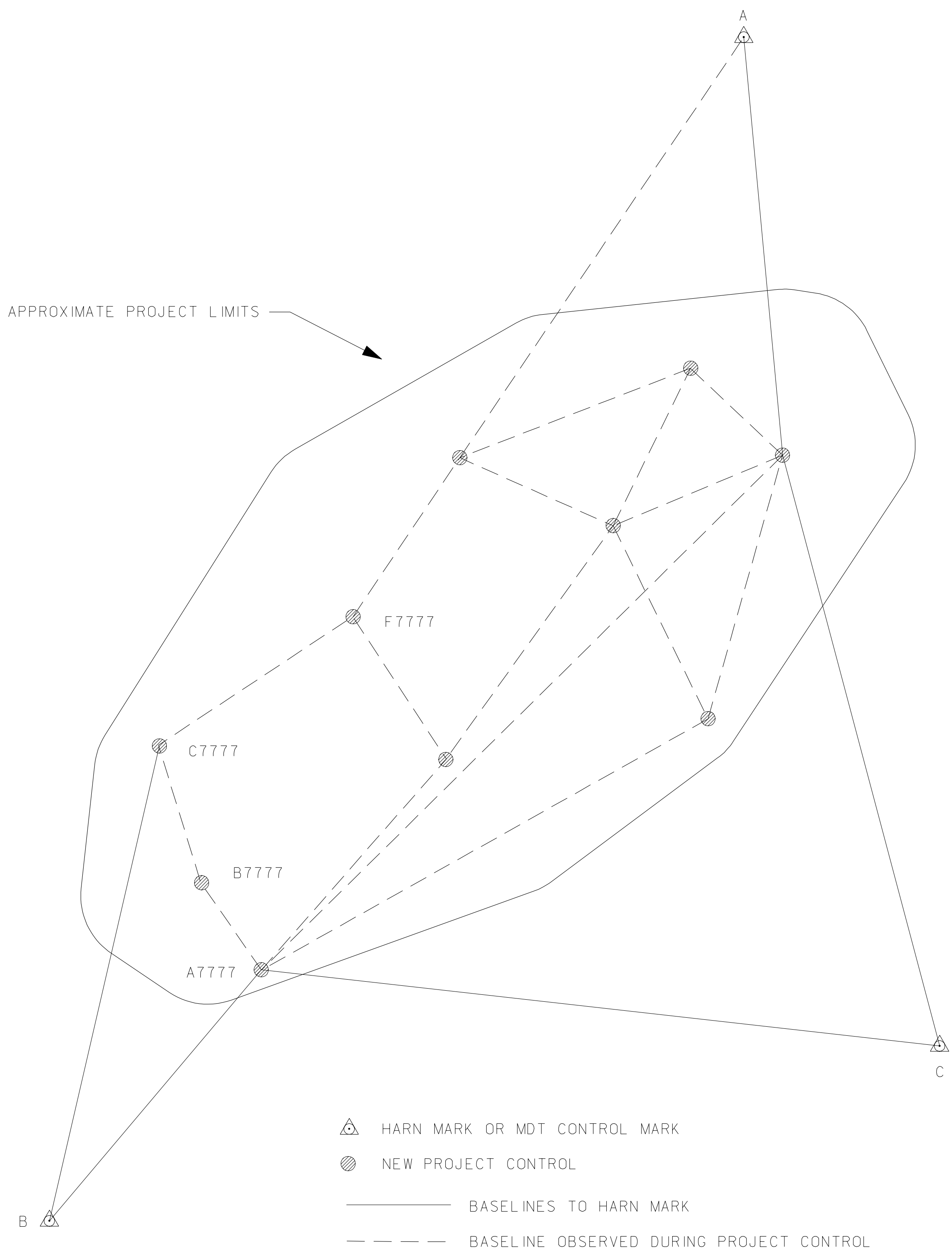


Figure 8-6
Network Configuration-Loop

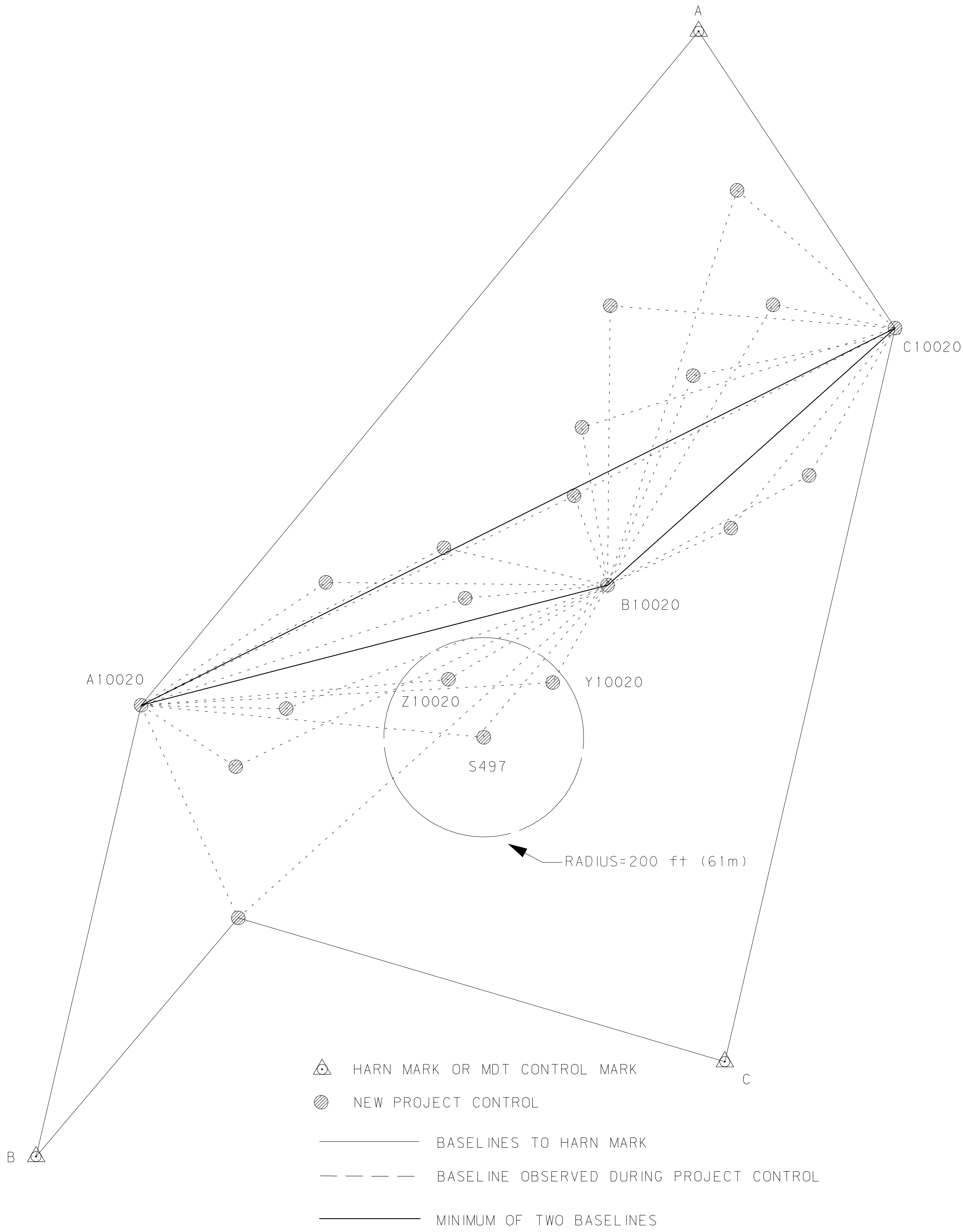


Figure 8-7
Network Configuration-Radial

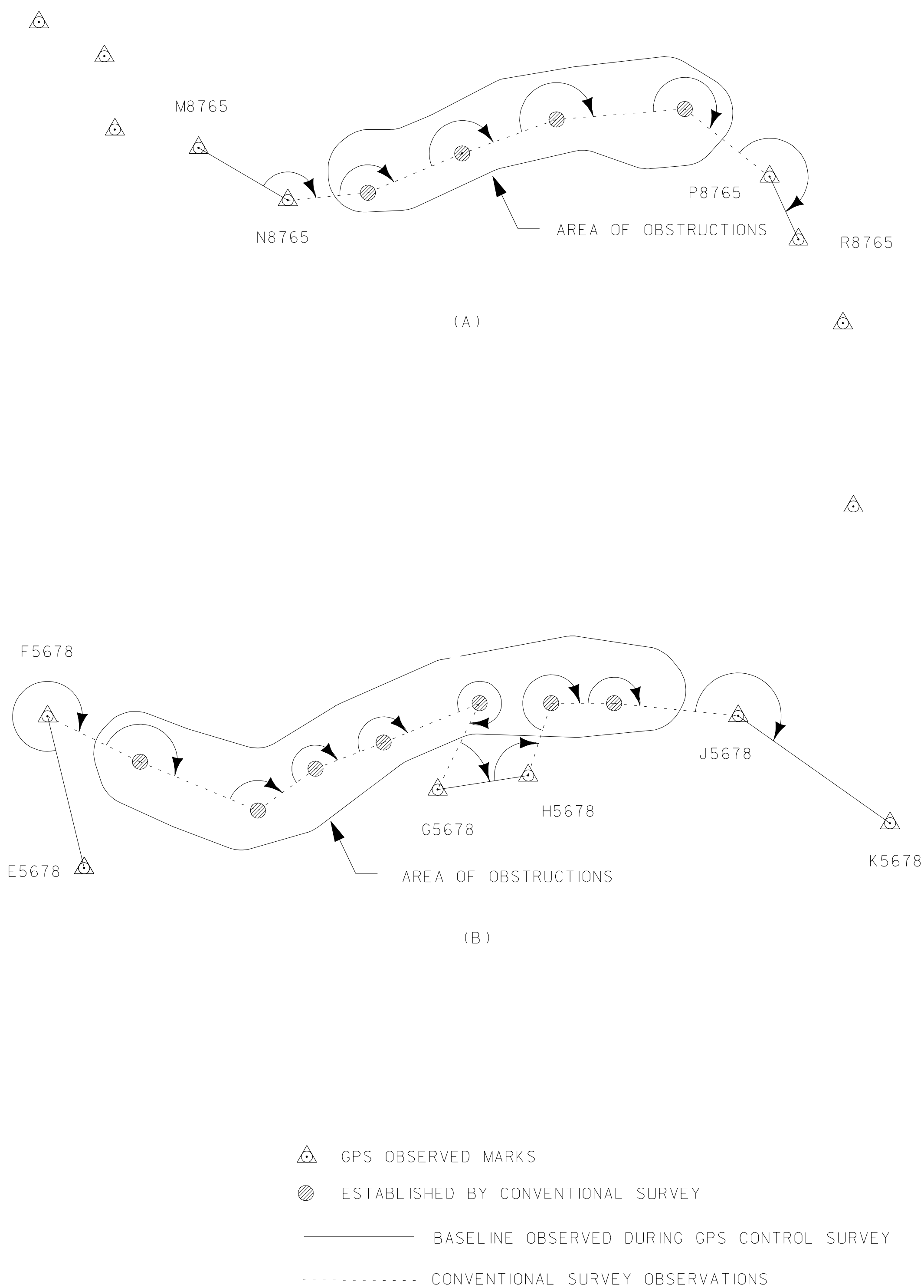
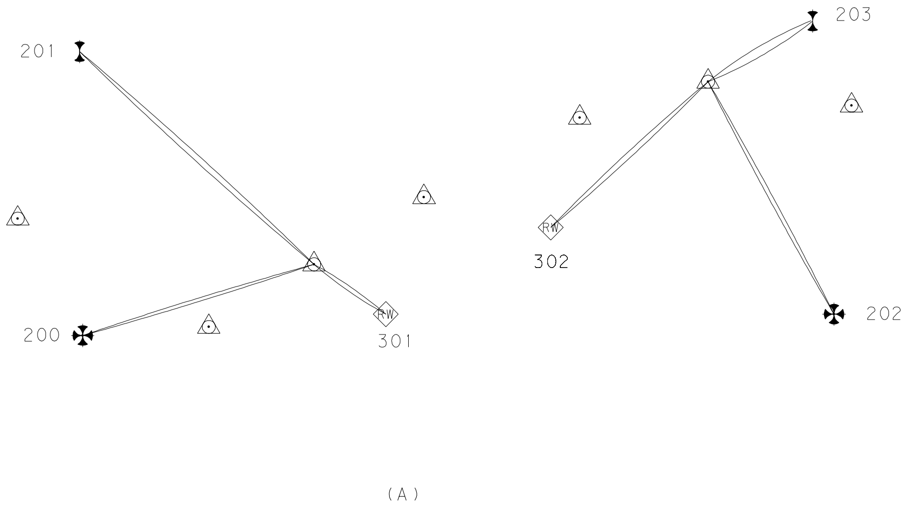


Figure 8-8
GPS Obstructions and Conventional Survey-Control



Minimum GPS Cadastral Observations

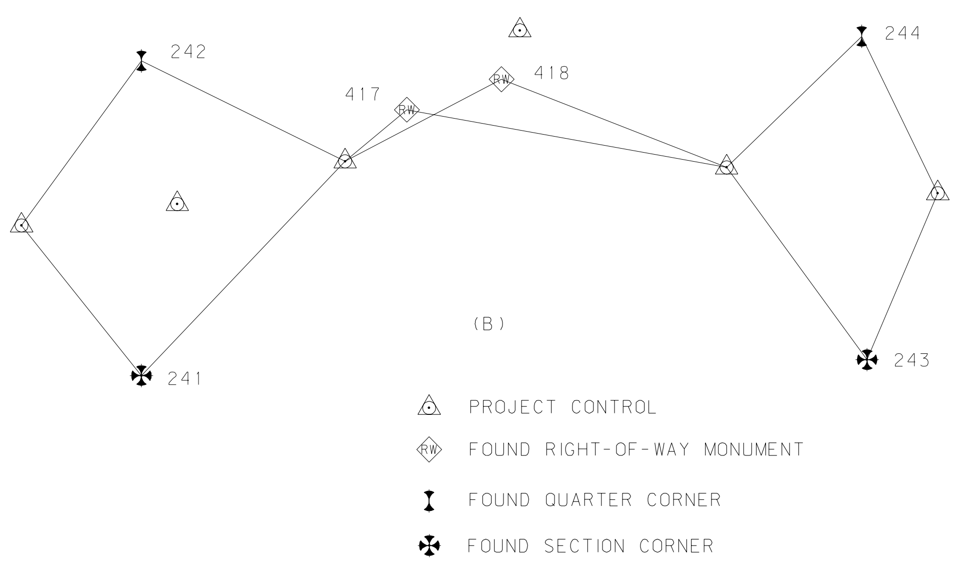


Figure 8-9
Recommended GPS Cadastral Observations

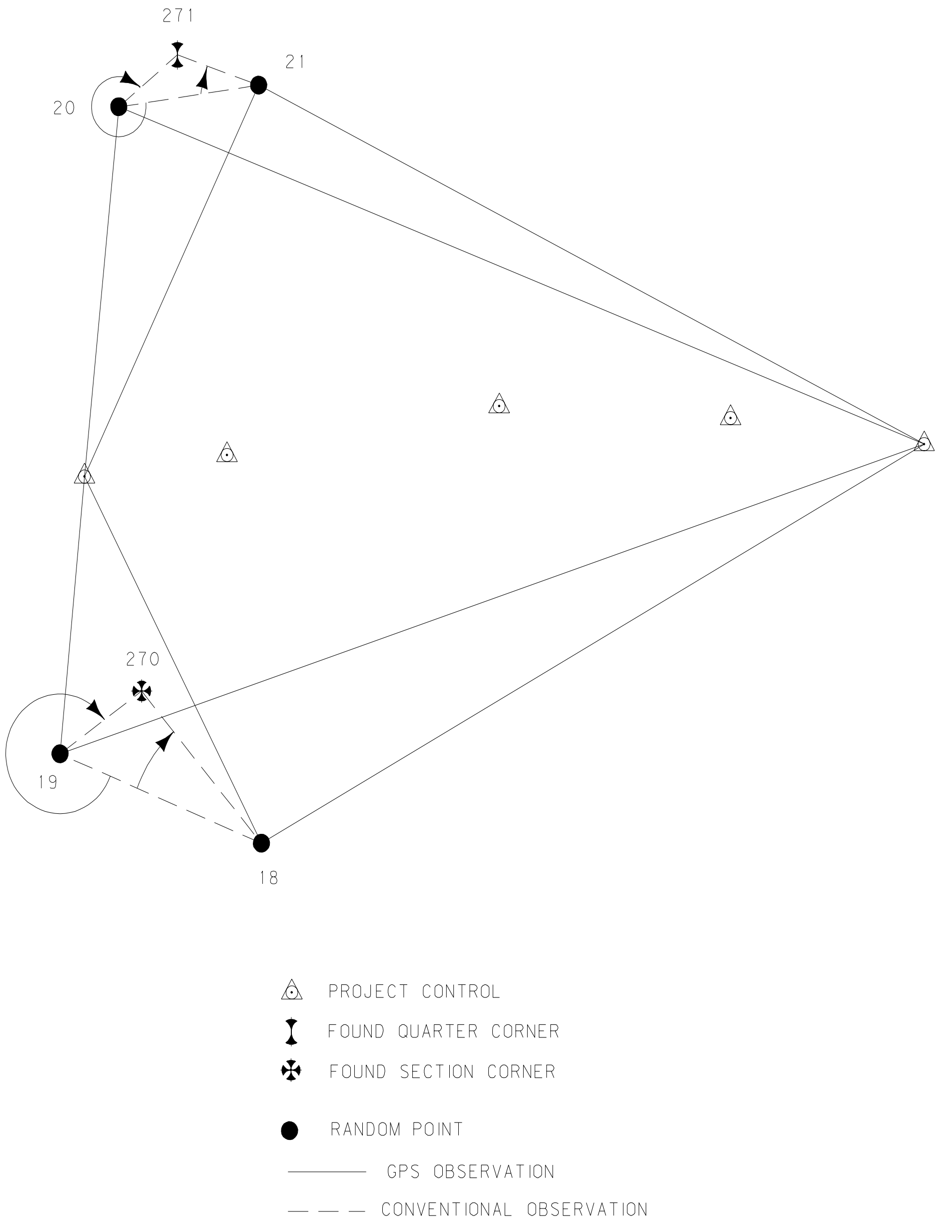


Figure 8-10
GPS Observations and Azimuth Pair

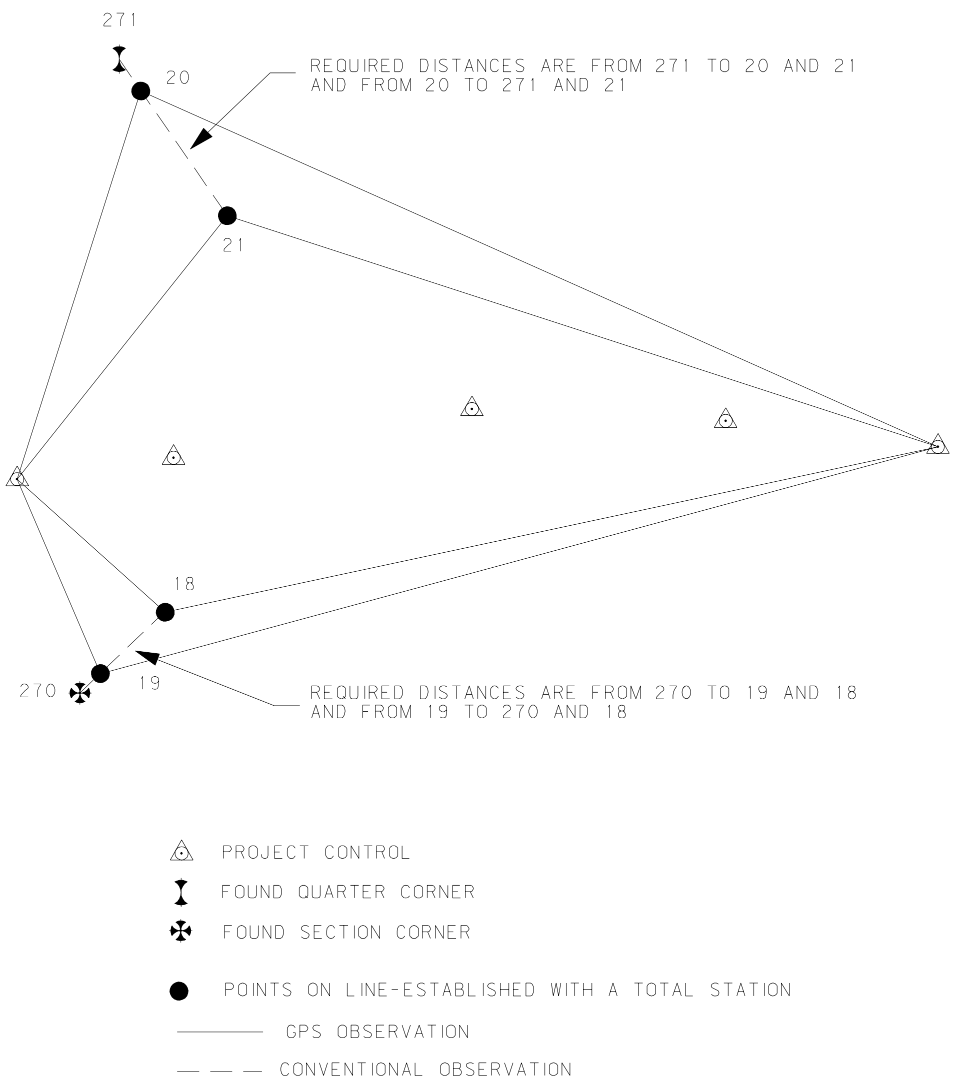
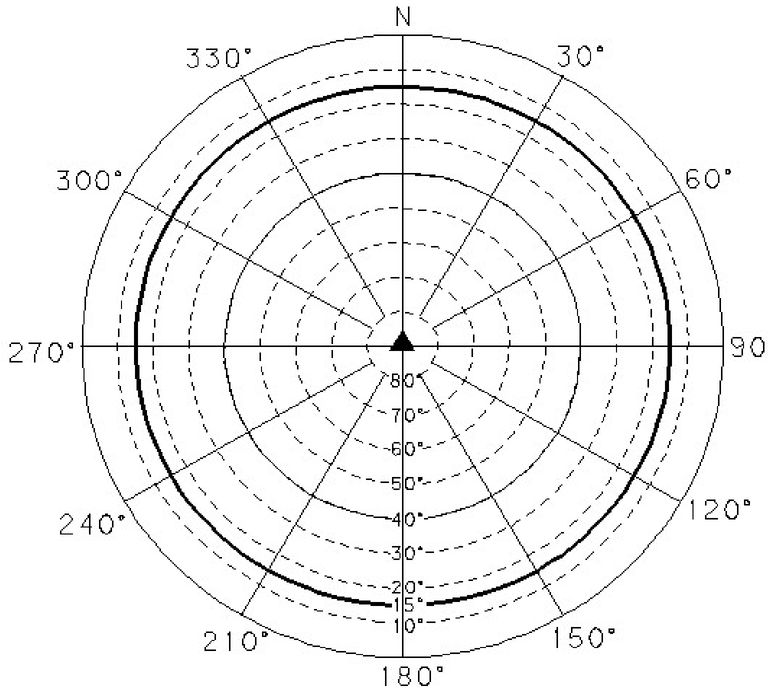


Figure 8-11
GPS Observations and Points on Line

#	HORZ ANGLE	VERT ANGLE
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		



INSTRUCTIONS;

Identify obstructions by azimuth and elevation angle (above horizon) as seen from Station mark. Include distance and direction to nearby structures and reflective surfaces (potential multipath sources).

STATION I.D.________CONTROL NO. ___________

DATE___________________________

DECLINATION (IF ANY) SET IN COMPASS______

NAME_______________ DISTRICT_____________

COMMENTS_________________________________

____

____

____

DECLINATION INFORMATION IS REQUIRED

CONTROL & PROJECT NUMBER____________________________DATE:____________________

Form 8-1
Station Visibility Diagram

RECEIVER NUMBER__________________FILE NAME________________________

LOCATION (PROJECT NAME)____

OBSERVER____

RECEIVER MODEL: (CHECK CORRECT MODEL) ☐ 4000 ☐ 4700 ☐ 5700

☐ OTHER SPECIFY)____

ANTENNA TYPE: (CHECK CORRECT ANTENNA TYPE)

☐ COMPACT L1/L2 W/ GP ☐ MICROCENTERED L2/L2 W/GP ☐ ZEPHYER GEODETIC

☐ OTHER (SPECIFY)____

GPS STATION NAME (MARK ID)	SLANT ANTENNA HEIGHT (M)	SLANT ANTENNA HEIGHT (FT)	LOCAL TIME START/STOP	RUBBING AND COMMENTS
			/	
			/	
			/	
			/	
			/	
			/	
			/	
			/	
			/	

MONTANA DEPARTMENT OF TRANSPORTATION

PAGE ________OF________

Form 8-2
Static/Fast Static Log

CONTROL NUMBER: 9999
PROJECT NUMBER: STPHS 9999(9)
Project Name: SOMEWHERE IN MONTANA-EAST & WEST

DATE: THIS DATE IS THE DATE OF THE OBSERVATION
JULIAN DAY: 159
SESSION: 1

ALL SESSIONS ASSOCIATED WITH THIS PROJECT ARE FAST STATIC OR STATIC

OBSERVER	RECEIVER# & TYPE	FILENAME	MARK ID	
WALT F.	61 (4700)	RV611591	X446	11:00 - 11:10
				11:15 - 11:25
			T446	12:00 - 12:10
				12:15 - 12:25
			A9999	12:50 - 1:00
				1:05 - 1:15
			D9999	1:20 - 1:30
				1:40 - 1:50
MIKE R.	62 (4700)	RV621591	103B	11:00 - 11:10
				11:15 - 11:25
			102D	12:00 - 12:10
				12:15 - 12:25
			C9999	12:50 - 1:00
				1:05 - 1:15
				1:20 - 1:30
				1:40 - 1:50
DAVE H.	71 (4700)	RV711591	AA9999	11:00 - 11:10
				11:15 - 11:25
			A9999	12:00 - 12:10
				12:15 - 12:25
				12:50 - 1:00
				1:05 - 1:15
				1:20 - 1:30
				1:40 - 1:50

Form 8-3
Observation Schedule

Control Number: 9999
Project Number: STPHS 9999(9)
Project Name: SOME WHERE IN MONTANA-EAST & WEST

MARK	LATITUDE (NAD83-92)	LONGITUDE (NAD83-92)	ELLIP	NORTH	EAST	ELEV	CSF	CONVERGENCE
102D (H1/V1)	46ø 54' 17.94369" N	114ø 1' 23.656072" W	987.565	304963.1120	255771.3501	1002.346	0.99923971	-3ø 18' 32"
103B (H1/V1)	46ø 53' 40.28923" N	114ø 0' 36.625738" W	983.557	303745.6319	256697.5703	998.325	0.99924069	-3ø 17' 57"
A9999	46ø 53' 14.40597" N	114ø 1' 36.155687" W	955.231	303020.7796	255394.1464	970.063	0.99924540	-3ø 18' 41"
B9999	46ø 53' 18.06544" N	114ø 1' 39.283314" W	954.420	303137.3504	255334.6064	969.248	0.99924548	-3ø 18' 43"
C9999	46ø 53' 13.08239" N	114ø 1' 40.880333" W	954.809	302985.7749	255291.9881	969.639	0.99924548	-3ø 18' 44"
D9999	46ø 53' 13.73085" N	114ø 1' 48.116371" W	953.550	303014.6018	255140.2980	968.387	0.99924567	-3ø 18' 49"
AA9999	46ø 53' 6.23164" N	114ø 1' 31.760247" W	956.276	302763.5574	255472.4233	971.100	0.99924532	-3ø 18' 37"
T446 (V2)	46ø 54' 20.93543" N	114ø 3' 7.814957" W	952.766	305182.8477	253577.3162	967.629	0.99924513	-3ø 19' 48"
X446 (H2/V2)	46ø 52' 30.71227" N	114ø 0' 17.052956" W	958.547	301578.1184	256987.5288	973.320	0.99924536	-3ø 17' 43"

Notes:
North East and Elevation are metric values
Elevations are GPS derived NAVD88
(H1/V1) Constrained horizontal and vertical per CN 4878
(H2) Constrained horizontal per CN 7785
(V2) Constrained elevation per NGS

Form 8-4
Coordinate List

Control Number: 9999
Project Number: STPHS 9999(9)
Project Name: SOMEWHERE IN MONTANA-EAST AND WEST

MARK	NORTH	EAST	ELEV
102D	304963.1120	255771.3501	1002.346
103B	303745.6319	256697.5703	998.325
A9999	303020.7796	255394.1464	970.063
B9999	303137.3504	255334.6064	969.248
C9999	302985.7749	255291.9881	969.639
D9999	303014.6018	255140.2980	968.387
AA9999	302763.5574	255472.4233	971.100
T446	305182.8477	253577.3162	967.629
X446	301578.1184	256987.5288	973.320

Notes:
North East and Elevation are metric values
Horizontal coordinates NAD83(1992)
Elevations NAVD88

Form 8-5
Coordinate List

Control Number: ______________________ Date: ______________________
Project Location: ______________________ Project ID: ________________
Coordinate System:
State Plane: _________ or Local: ______________
Ref Elevation: ___________(F)(M)
Base Receiver (Manufacturer & Model): ________________________________
Base Antenna (Manufacturer & Model): ________________________________
Base Receiver File Name:____
Rover Receiver (Manufacturer & Model): ______________________________
Rover Receiver (Manufacturer & Model): ______________________________
Rover Receiver File Name: ____
Base at: ____________ Start Time: ______________
Height: _____________(F)(M) End Time: _______________

Rover Observations:

Mark ID	*	Height of Antenna	Description

* Check if a Calibration Mark Page 1 of _____

Form 8-6
Kinematic Observation Log Page 1

Rover Observations:

Mark ID	*	Height of Antenna	Description

* Check if a Calibration Mark Page ____ of _____

Form 8-7
Kinematic Observation Log-Continued

Chapter 9

Reserved for Data Collectors

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Chapter 10

Construction Surveys

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Chapter 10
Construction Surveys

10.1 GENERAL

This chapter of the Surveying Manual suggests and explains procedures and techniques that promote correct, thorough and uniform construction staking and measuring.

10.1.1 PURPOSE

The purpose of construction surveys is to establish the required horizontal and vertical control marks required for the construction of a project. This includes items such as taking basic measurements necessary to determine quantities, and then documenting these measurements to substantiate the final payment made to the contractor. Construction staking establishes basic line and grade controls, delineates working areas, and serves as a base to verify the locations and quantities of completed work. Normally, the Montana Department of Transportation (Department) provides the essential controls for establishing lines and grades, and the contractor's forces set supplemental stakes for their convenience, particular method of operation or specific equipment. In some cases, such as for large structures, the contractor may set most of the control points.

10.1.2 SURVEY METHODS

The increased use of computers and specialized design equipment has led to many advances in the entire highway engineering process. The changed emphasis on computer use and special graphic systems in the location and design process mandates that the preliminary survey be based on a three-dimensional coordinate system.

The benefit of this system is that preliminary survey data and monumentation can be planned for and used to advantage in the construction surveying process. If all control points of the construction plans are tied to a coordinate system, the position of any point, or any references to the point, can be calculated. Calculations can be made with hand-held calculators, personal computers, total stations, or data collectors.

The primary method of staking is from known control points established during the preliminary survey by measuring of angles, distances, and elevation differences. This method is referred to as radial staking and can meet required accuracy if sound surveying practices are used. Radial staking may be used for staking catches and cross sections. Establishing elevations for other purposes may require differential levels.

The computations for radial staking include a backsight to a visible control point, azimuths, distances, and differences in elevation (if required) to all points to be set within the specified distance range of 1000 ft (300m) from the occupied control point.

A secondary method of staking is the traditional line-reference method. This method is used for establishing construction control stakes in conjunction with radial methods. An example of this type of staking would be setting intermediate control points on a centerline or a reference line using radial methods and then using traditional line, distance, and/or deflections for laying out individual stations.

10.1.3 MANAGEMENT OF ENGINEERING CONSTRUCTION SURVEYS

Good management practices are equal in importance to good engineering practices in reducing costly engineering control errors in highway and bridge construction.

The construction survey accounts for a large proportion-historically 30 percent-of the construction engineering costs. Therefore, special attention is needed to promote efficiency and economy. The District or consultant shall schedule surveys, provide the manpower to staff the construction survey crews and ensure that efficient practice is maintained at all times. The Engineering Project Manager (EPM) or consultant should have full knowledge of the construction survey work and of the proper methods to be used to ensure surveys are performed properly.

10.1.3.1 Assignment of Survey Personnel

It is desirable to assign the same personnel to stake the project that performed the preliminary survey. It is generally accepted that more interest is shown and more care taken in survey work when the crew members know that they will be assigned the construction of the facility for which the survey is being made. Personnel—either temporary or permanent—new to highway and bridge staking should be carefully trained and observed to assure that proper surveying practices are followed.

10.1.3.2 Planning

Whatever method of staking is used, planning and effective use of personnel and facilities are essential in providing the most economical survey. Much of the planning, such as layout of horizontal and vertical control, should have taken place during the preliminary survey. Establishment of additional control points (control densification) may be necessary for construction staking. Refer to Chapters 5 and 6 for conventional survey methods and Chapter 8 for Global Positioning System (GPS) methods. If it is apparent that any control monument will be destroyed during construction, a new point shall be set or the existing point shall be referenced.

Generally, a lot of time passes, and often crew changes occur, between the establishment of the original control survey and the use of that control for construction staking. The survey crew should thoroughly review such items as design plans; horizontal and vertical control listings, and all other design output that could aid in planning the survey. A field review and re-flagging of control points should also be conducted.

Preparing staking notes or data collection files in the office whenever possible saves costly survey crew time and often prevents field-calculation blunders. Before preparing staking notes or data collection files, review all computer listings thoroughly and verify that the output is based on the same design as shown in the construction plans. Void old information not applicable to the construction plans. Review all design information for accuracy and thoroughness before any construction staking procedures commence.

Allow sufficient time for checking plans, staking, referencing, and checking of primary controls prior to the beginning of construction. Consideration shall be given to expected weather conditions, availability of points, terrain, and the required accuracy of the survey.

Personnel should have an opportunity to review the plans and become familiar with construction details affecting the layout. Plan layout and referencing to keep duplication of work to a minimum. The time spent on planning, prior to layout, will save valuable time during construction.

10.1.4 SAFETY

Construction projects can be hazardous working areas because of the concentration of heavy equipment, dusty conditions and, in some cases, normal highway traffic.

Total stations and levels may frequently be set up in construction zones and/or near rapidly moving traffic.

The safety of the traveling public, construction crews, and MDT workers is important. The Department has established safety and signing standards for use in the work place. Refer to Chapter 1 and other relevant documents.

10.1.5 STAKING AUTHORIZATION

Construction staking is normally authorized when the project is advertised for bids-two to three weeks before the bids are opened. Frequently, the district construction engineer may wish to begin staking complex projects prior to the advertising date.

Authorization to begin construction surveying prior to contract award can be obtained from the Construction Bureau. On such occasions, the District Construction Engineer (DCE) should draft a request stating the reasons for early staking and submit the request to the Construction Bureau. If staking is approved, the Construction Bureau will obtain the proper charge numbers and send them to the DCE with the authorization.

10.1.6 CONSTRUCTION SURVEY DUTIES

Construction survey crews may perform many different functions in accordance with contract documents. Some of the more important of these may include:

- staking and referencing centerline
- checking, setting or reestablishing bench marks
- checking plans, grades and calculations
- performing grade revisions
- staking culverts and checking culvert lengths
- staking cross sections and setting slope stakes
- laying out interchanges (including structures), ramps and frontage roads
- checking finished grade stakes for subgrade, base surfacing and finished surfacing
- preserving, perpetuating and referencing of survey monuments
- recording all pertinent survey information in a neat, legible manner in accordance with the requirements presented in this manual

- providing practical and accurate survey controls in an economical manner

It is important that the majority of a survey crew is skilled and precise in their duties, but it is also necessary to exchange this information, to communicate knowledge of survey skills to other less informed members of the crew to promote and maintain the degree of teamwork necessary for efficient surveying. It is important that the various crewmembers exchange their knowledge, help each other, and learn the more important details that control the work. This exchange of knowledge and skills provides opportunity for advancement and provides the EPM with a team in which any one of the members can substitute for a member who is not present on any particular day. When possible, each crewmember should be provided an opportunity to perform different survey functions.

EPM and others, who are assigned to construction surveying duties, whether for entire projects or selected types of surveys, should acquire information and equipment necessary for the survey crews to perform their duties.

10.1.7 CARE OF SURVEYING EQUIPMENT

The EPM is responsible for all assigned equipment. Refer to Chapter 3 for information associated with the care and handling of surveying equipment. Any individual using survey equipment, especially instruments, should make every effort to prevent their being damaged.

If damage was caused by the contractor’s employees or equipment, documentation should reflect this information and the contractor should be advised so the matter may be referred to the contractor’s insurance company. Repair and replacement costs are often recoverable from contractors or motorists who have caused damage to instruments. The letter of transmittal accompanying damaged instruments should have a detailed explanation so others can understand how the incident occurred to expedite recovery of repair costs.

The Department may attempt to recover costs to repair or replace instruments damaged by contractors or motorists.

10.1.8 CHECKING PLANS

Review all plans, contract documents and applicable agreements for the project before beginning construction-staking activities. Resolve any found discrepancies before staking of that item.

Make notations for any additional requirements needed to serve new properties that are not indicated on the plans but which may have been in existence at the time right of way was secured. Make no additions, deletions or revisions to any entrances, condemned property, or to controlled access right of way without the approval of the District and the Construction Bureau.

10.1.9 COOPERATION WITH CONTRACTOR

Staking is a very critical operation on projects where contractors employ modern, high-production equipment and methods. Detailed planning and intimate cooperation with the contractor are required so staking can begin as soon as is reasonably possible, and as far in advance of construction as weather and soil conditions will permit. Adequate lead-time is essential to prevent delaying the contractor's operation. Staking delays that hinder the construction operations should be avoided.

The EPM should consult with the contractor as soon as possible after the project is awarded to coordinate the staking plans with the contractor's schedule. These discussions may be held before the preconstruction conference, and, if so, the plans should be reaffirmed at the preconstruction conference. Contract documentation may require that the contractor furnish a schedule of operations and keep it updated to allow determination of work force and equipment needs for timely staking.

Explain the method of staking, how the stakes are to be marked and guarded, what offsets will be used, etc. The information placed on stakes, abbreviations, and locations of messages, if not carefully explained to contractor personnel, may result in misunderstandings or misinterpretations and delays.

The need to retain referenced control points and the possible cost to the contractor in terms of stake replacement and work delay awaiting new stakes should be discussed with the contractor. Contract documentation provides the contractor be held responsible for preservation of all stakes and marks. If construction stakes or marks are carelessly or willfully destroyed by the contractor, and it is necessary to replace stakes removed by the contractor's operation, sufficient documentation should be kept, including the date replacement began, personnel assigned, and the reason for replacing the stakes.

The contractor shall give notice of the need for staking, elevation checks, etc. sufficiently in advance to permit timely staking and checking and efficient utilization of project personnel. Procedures for this should be established at the preconstruction conference.

10.2 HORIZONTAL CONTROL

10.2.1 GENERAL

The construction phase of most projects requires a relatively dense network of horizontal control monuments. This network normally consists of the control monuments that were established during the preliminary survey and additional control monuments established specifically for construction purposes. It is essential that the control used to design the highway be used to construct the highway.

10.2.1.1 Densification of the Existing Control Survey

During the construction of a project, additional control monuments may be required. New control marks should be established in conformance with the methods and procedures described in Chapters 5, 6 and 8. With proper planning, much of the necessary control will be included in the original project control survey that was completed during the preliminary survey.

A thorough review of construction plans and analysis of staking requirements generally indicates where additional control is required. Interchanges, structures, or other complex facilities usually require monuments in unforeseen locations. Some basic suggestions for establishing additional control monuments are listed below.

- Set control points where they will be most useful for staking.
- Set control points where they are accessible by vehicle, if possible. A vehicle can often be used to shield the instrument person from wind. In addition, hand-carrying equipment to an inaccessible point is time-consuming.
- Pick locations where the instrument is above all stages of the work.
- Flag and set witness posts next to the new control points so that they are easy to find and will not be disturbed by construction.
- The additional control points shall not duplicate a number used in the original control.

Reference and reset any control mark established during the preliminary control survey, if construction activities will destroy the point. Refer to Chapter 5 and Figure 5-3, Control Point Referencing. Resetting of original control marks may eliminate or reduce the need to establish additional control points.

After the completion of the project, update the control diagram on the as-built plans to reflect additional control marks and any reset control marks. This update should include the point name (as stamped on the aluminum cap), northing, easting, elevation, and location. After construction is complete, leave enough control on the project so that the elements of a control network exist.

10.2.1.2 Construction Staking

Two basic staking methods are used to establish survey marks required during the construction of a project. They are radial staking or traverse staking. The radial staking method is preferred.

Radial staking involves the use of inverse calculations that give azimuth and distance, and, where required, elevation from the control monument to the construction stake. If several stakes are to be set from one control point, the backsight setting should be re-checked during the setup and at the end of the setup to ensure instrument orientation was not disturbed. The data collector simplifies this method.

Traverse staking is accomplished by running a traverse through the points to be staked, and setting the points as the line is run. Control points on the line are established from the control survey by radial staking. The control points on a centerline are Point of Curvature (PC), Point of Tangent (PT), Tangent to Spiral (TS), Spiral to Curve (SC), Curve to Spiral (CS), and Spiral to Tangent (ST). Once the control points for the line to be staked has been established, the remaining portions of the line can be staked by traverse from these control points.

Radial Staking Radial staking from the preliminary control survey has advantages, including the following:

- The entire centerline and other required points can be set from the control monuments established from the preliminary control survey.
- Sections of the project can be established from a single instrument setup.
- Random points can be set where they will not be disturbed by construction activities.
- 3D coordinates of stakes can be established making full use of total station capabilities.
- The same control point can be used for setting all phases of construction, including the restoration of stakes obliterated during construction operations.
- Distance, direction, and elevation calculations can be pre-figured from any control point to any desired construction stake, or such calculations can be made by simple inversing, using hand-held calculators or data collectors.

Right-of-way monuments, centerline references, and pipe or structure reference points shall be set from control points using the radial stake out method.

Traverse Staking Traverse staking has some advantages over the radial staking method. The traditional system of instrument setups at control points, sighting on line, or turning appropriate angles to set points may offer some advantages such as:

- straightforward
- well known by construction people
- provides actual on-the-ground checks of centerline
- requires fewer computations
- at times may be faster
- fewer computations since angles required for curves and chord lengths can be calculated in the field
- provides on-the-ground and visual checks of the centerline

Some disadvantages of the traverse staking method are:

- requires additional instrument setups
- adapts poorly to irregular terrain
- may be slower than radial staking
- may require a larger crew

In addition to the above disadvantage, if the new alignment follows an existing alignment, this system may present some overriding disadvantages:

- Instrument setups on pavement often require special treatment, such as traffic control.
- Increased costs and personnel required for crew protection.

10.2.2 ESTABLISHING CENTERLINE

Establishing the centerline or designated project baseline may be one of the first procedures in staking a highway project. It is obvious that any stakes set on the centerline will be destroyed during construction operations. The centerline or designated project baseline may only need to be temporarily marked for use during staking.

Check the plans and prepare the notebooks before beginning staking and at least one set of plans is to be available for use by the survey crew at all times. Occasionally, errors in the plans or the original field location are found by the survey crew. Notations of these errors are to be made on the field plans as well as in the notebooks.

All centerline control and critical alignment points, including superelevations control points, will be set directly from the control survey points using radial staking methods. Once the centerline control is set, the remaining stations can be set using radial or traverse methods. Select an option based on the personnel and equipment available, terrain, length of time needed, and safety.

Centerline stakes are typically set between the centerline control, on even 100-foot (30m) stations on tangents and every 50 ft (15m) on curves or as the terrain dictates.

10.2.2.1 Centerline Reference Points (RPs)

The cost and time required for resetting stakes, or for setting new lines of construction control stakes, can be reduced by setting reference monuments outside the construction limits before construction is started. If centerline RP's are established, they will be set using either radial survey.

Consultant Design Projects The contract between a consultant and the Department may require that the consultant is responsible for the survey and reference of the final design centerline. If this has not been done, the EPM should notify Consultant Design.

Centerline reference points, if used, should be set three feet inside the right-of-way line only if the right-of-way limits are known. Existing fences cannot be considered the right-of-way line. If right of way is unknown, the centerline reference points shall be marked to avoid confusion with reference points for the right-of-way. RP's are generally set at the major control stations associated with the centerline, for example PC's and PT's. References may also be set at specified distances along tangents.

RP's will consist of a 5/8-inch x minimum 24-inch (16mm x 600mm) rebar with a red plastic cap. The caps should be at or near ground level. Set guard stakes approximately 6 to 8 inches from the RP.

10.2.2.2 Centerline Re-establishment

When centerline stakes are re-established, they may be set from the RP's, from the control survey established during the preliminary survey, or from the additional control marks set during construction.

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10.3 VERTICAL CONTROL

10.3.1 GENERAL

Uniform vertical control is an important part of all engineering projects. A relatively dense network of vertical control, (bench marks), shall be established for most projects prior to construction staking operations. The establishment of vertical control is seldom accomplished in one survey; it usually is a culmination of several vertical surveys beginning with the bench levels associated with the preliminary control survey. The most important aspect of the various stages of vertical control is that the same datum is used from preliminary surveys through design and final construction control.

10.3.2 ESTABLISHMENT

Ideally, most of the project control marks are established during the preliminary stage of project development. This existing network is then densified by differential levels throughout the preliminary and construction phases.

If there has been a long delay between the time that the preliminary survey was conducted and the design plans were published, field check the condition of the monuments. Check each monument for possible disturbance or settlement. If a monument is in satisfactory condition, re-flag it for ease in location.

Review design plans thoroughly, and mark up a set of plans with the approximate location of existing control and locations where additional construction bench marks (BMs) are required. Marking up the plans and flagging of control in the field can save considerable field time and assure that required BMs are established at their optimum location on a timely basis.

During the planning process above, determine the bench lines to be established for the initial grade staking (normally earthwork on main lines and service roads), final grade staking, structure staking, and vertical control monumentation. Plan each bench line to reduce the difficulty and length of level runs required to establish the subsequent bench marks.

Check levels should be run on existing vertical control. If errors are found, corrections should be made and new descriptions for the bench marks established and recorded in the level book with the proper cross references to the original level data. Bench marks that will be disturbed during construction should be replaced outside construction limits and the proper records made.

10.3.3 VERTICAL CONTROL LOCATIONS

The majority of the vertical control will be established during the preliminary survey phase. The required density of vertical control depends on terrain, vegetation, and type of construction. There should be enough control to decrease the survey time for subsequent leveling requirements. The advantage of density against the greater initial cost for establishing extra bench marks must be considered. Additional BMs should be established in locations suitable for the purpose and for permanence.

Vertical control should always be established outside the construction limits. Vertical control that were established during the preliminary survey and are in the way of construction should be reset outside the construction limits. Vertical control should be set in accordance with Chapters 4 and 5.

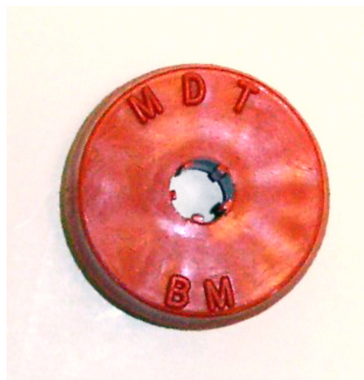
The height of fill and depth of cut should be taken into consideration when establishing BMs so locations are convenient for future surveys performed before, during and after construction of the roadbed.

10.3.3.1 Bench Marks

BMs that are to remain as reliable elevation references over a period of years, or even for extended construction periods such as for major structures, should generally meet the following criteria:

- They should be placed in stable, undisturbed original ground.
- They should be established using the requirements and tolerances given in Chapters 4 and 5.
- They can be placed on abutments or wingwalls of older existing structures that are stabilized.
- They should be placed at points of general elevation with the adjacent terrain. A BM on top of a high bluff is not as desirable as one below the bluff, provided all other criteria can be met. Quite often, the positions of control points are not compatible with project use, and BMs should be established in more usable locations from the control.
- All bench marks should be described in detail as to location and type of bench mark.
- Project BM are typically rebar with an aluminum cap or a Morasse® P43 plastic marker drove over the rebar.

Project Bench Marks The density of vertical control in the project area can lead to confusion and possible errors because of misidentification. All project bench marks will be established in accordance with Chapter 4. Identify each bench mark uniquely by name, number, or location. During periods of use, a flagged or painted lath can help in locating each bench mark quickly. If additional bench marks are set, they should consist of a 5/8-inch x minimum 24-inch (16mm x 600mm) rebar with a Morasse® P43 (available from Berntsen Survey Markers) or equivalent. When ordered from Berntsen the P43 should be pre-marked "MDT BM". The BM number shall be stamped on the cap.



Morasse Monument

Temporary Bench Marks Less-permanent bench marks may be required for a limited-use period or for a specific survey operation—for example, slope staking. TBMs may be 2-inch by 2-inch wooden hubs of sufficient length to be stable when placed in the ground; a painted or chiseled mark on concrete or rock outcrops. Temporary bench mark accuracy should be consistent with the type of construction for which the TBMs will be used.

10.3.3.2 Vertical Control Tolerances

The maximum error in establishing or checking bench marks is outlined in Chapter 4. However, an error of 0.05 ft (0.015m) is allowable in level loops for such work as cross-sectioning, setting grade stakes and slope staking. Acceptable readings of the level rod are shown below for each type of work:

- 0.01 foot (0.005m) for setting grades for forms, structures, pavements, curbs, gutters, walls, rails, etc
- 0.05 foot (0.015m) for setting subgrade stakes (bluetops)

- 0.1 foot (0.05m) when reading ground elevations for cross sectioning and setting slope stakes

Refer to the current version of the contract documentation for finishing, measuring and staking requirements. If there is a difference, then the contract documentation shall prevail.

10.4 EXISTING MONUMENTATION

Highway construction frequently will destroy property controlling monuments, property monuments, National Geodetic Survey (NGS), or United States Geological Survey (USGS) bench marks. Steps should be taken to preserve or reference all existing monuments. This perpetuation may require the preparation of a certified corner recordation form. If possible, existing monuments should be preserved during the course of construction operations. This may eliminate the preparation of corner recordations.

The contract documentation or right-of-way agreements may specify who is responsible to preserve, reference, or file corner recordations for existing monuments. If it is the contractor's responsibility, the Engineering Project Manager (EPM), the contractor, and the contractor's land surveyor shall coordinate with the District's land surveyor prior to the destruction and/or perpetuation of any monument. If it is the Department's responsibility, the EPM shall coordinate with the District land surveyor.

10.4.1 PERPETUATION OF PROPERTY CONTROLLING MONUMENTS

All property controlling monuments (section corners, quarter corners, etc) that will be disturbed by construction will be referenced prior to construction, and a corner recordation form showing the references will be prepared and recorded. After construction has been completed, the location of the destroyed property controlling corner will be reestablished with a monument that conforms to the requirements given in Chapter 6, and a new corner recordation will be prepared and recorded. The contract documents will indicate if this is the responsibility of the contractor or the Department.

10.4.2 PERPETUATION OF PROPERTY CORNERS

Right-of way agreements made between the Department and individual owners may specify the location of existing property corners near the PTW will be preserved. In this case, a corner recordation will be prepared (including reference monuments) and recorded for each existing property corner that will be destroyed by construction. Agreements stipulating new monuments will be established on the new right-of-way will be conducted per state statutes and the Administrative Rules of Montana. Contract documents should indicate if this is the consultant's responsibility or the Department's responsibility.

10.4.3 RELOCATION OF NGS AND USGS BENCH MARKS

Refer to procedures outlined in Chapter 2 for re-establishment and relocation of NGS and USGS bench marks.

10.5 RIGHT-OF-WAY MONUMENTATION

10.5.1 GENERAL

The right-of-way plans and corresponding deeds of each right-of-way parcel shall be used to ensure that right of way, as shown on the plans is compatible with the corresponding deed. Should assistance be necessary, the Right-of-Way Bureau should be contacted. State law requires right-of-way monumentation be conducted under the direct supervision of a professional land surveyor (PLS) licensed to practice land surveying in the state of Montana.

10.5.2 RIGHT-OF-WAY MONUMENTS

Right-of-way monuments are to be 5/8-inch x minimum 24-inch (16mm x 600mm) rebar with a 2” (51mm) MDT aluminum caps. The cap is stamped with the land surveyor’s registration number, the year set, RW, and if available the right-of-way point number as shown on the right-of-way plans (i.e. RW 2000). Right-of-way monuments are set on the right-of-way at changes in the right-of-way width, at the beginning and end of projects, and at the control points associated with the horizontal alignment or the right-of-way baseline. The right-of-way base line may not be the same as the design centerline.

10.5.2.1 Establishment of Right-of-Way Monuments

All right-of-way monuments will be set using radial survey methods and the control marks associated with the control survey. Right-of-way monuments may be set using conventional instruments such as total stations and field notes, data collectors and total stations, or GPS. In all cases, the monument will be set and then its position will be verified using an independent observation. Examples of conventional survey methods to verify the position of a set monument would be:

- An additional radial tie is made from a different control point to the set monument.
- An additional radial tie is made from the same control point with a different backsight to the set monument.
- An additional radial tie is made from a random point to the set monument.

Refer to Chapter 6 for a detailed explanation of approved conventional radial survey methods and Chapter 4 for coordinate tolerances.

Additional methods to obtain an independent observation would be:

- The monument is set using conventional observations and then tied from a different control mark using GPS.
- The monument is set using GPS and then tied using conventional methods.
- The monument is set using GPS and then an independent GPS observation is obtained.

Regardless of the methods used, a coordinate list will be prepared that compares the designed coordinate with the position as staked. The radial differences must be equal to or less than 0.25 ft (0.076m). In addition, all digital files associated with the establishment of right-of-way surveys will be placed in the project folder using DMS. If right-of-way is staked by a consultant, this information will be provided to the appropriate area for DMS download.

10.5.2.2 Right-of-Way Equals Existing

Right-of-way plans may indicate locations where the newly acquired right-of-way intersects the existing right-of-way. A right-of-way monument will **not** be set at this point. Nor will right-of-way monuments be set on any line indicated as existing right-of-way, unless a retracement of the existing right-of-way in the area has been completed by a PLS licensed in the state of Montana.

10.5.2.3 Reference Monuments (Optional)

Reference monuments for the right-of-way, if set, are to be 5/8-inch x minimum 24-inch (16mm x 600mm) rebar with a standard MDT red plastic cap. The reference monument is set 3 ft (1m) inside the right-of-way monument (between the monument and centerline) on a line perpendicular to tangents or 90 degrees to a line tangent to curves.

10.5.2.4 Witness Post

Witness posts are required, except where they would infringe on private property or conflict with urban facilities. A Department witness post and right-of-way decal shall be set 1 ft (0.3m) inside the right-of-way monument.

10.6 CLEARING AND GRUBBING STAKES

10.6.1 GENERAL

The contract documentation provides different methods of measuring clearing and grubbing units: a lump sum basis, an area basis, and areas to be included in the cost of other items. Staking shall be compatible with the method of measurement. Clearing and grubbing limits are established on the plans. Sharp breaks in the width of the clearing line may be avoided by adjusting clearing stakes; assuring that the minimum Clear Zones are maintained. Clearing lines on the inside of curves and at intersections should be given special attention to provide adequate sight distance and clear zone requirements, as per the MDT Road Design Manual, when the right-of-way widths will permit.

Distances will be measured to the nearest foot (0.5m), and stakes or laths will be placed to clearly distinguish the intended limits. Stakes or laths should be spaced approximately 100 ft (30m) apart, as a rule, and placed so one lath can be seen from the next one. Spacing will depend on the terrain and denseness of the foliage. When clearing areas of heavy timber, clearing stakes should be set in a somewhat irregular alignment so when clearing is completed a more natural appearance will result. Stakes or lath properly set are marked “clear” and are usually flagged.

The stakeout notes will show stationing, distance left and right, date, and names of persons making the stakeout. The areas that have been staked should be reviewed with the contractor’s supervisors so they are aware of the staking procedure and the general areas to be cleared or grubbed, and so they can raise any questions before work begins.

A limited number of trees and shrubs may be retained to blend the right-of-way with adjacent terrain, providing screens for unsightly areas, and enhancing the natural appearance. Mark the trees to be saved with a red cloth or other suitable identification and inform the contractor that trees so marked are not to be cut. Trees and shrubs to be retained should be sound, vigorous, and of a desirable species that will not create future maintenance problems.

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10.7 EARTHWORK QUANTITIES

10.7.1 General

Plan pay quantities are calculated from preliminary field cross-sections or from cross sections derived through the photogrammetric process. Normally, the staked location and elevation should agree closely with the plotted location and elevation.

10.7.2 Conventional Cross-sectioning & Slope Staking

The Department utilizes the "Average End Area" as one method of measurement for Earthwork Quantities. This method incorporates the template data for the specific station, the existing ground at that station and the slope catches to define a closed area. This information, when compiled with the other station areas, comprises the Earth Work Quantities.

10.7.2.1 Planning

All preliminary work such as centerline staking, establishing bench marks and offset lines and preparing a grade book or a system of grade computation sheets will greatly aid the survey crew. Cross sectioning will usually be performed when slope staking.

Slope staking and measurement of earthwork quantities are areas where more disputes with contractors arise than any other phase of highway work. Therefore, it is recommended that the EPM carefully plan and perform sufficient preliminary work in order to permit cross-sectioning work to follow in an orderly manner.

10.7.2.2 Cross-Section Intervals

Collect data at sufficient intervals to adequately represent the terrain. The EPM shall see that sufficient measurements are taken at proper locations, to truly reflect the quantities of earth to be moved. Bear in mind that a recorded cross section is the basic information for determination of contractual pay quantities and that the contractor and EPM need a certain quantity of stakes, properly located, for construction of the roadway to specified dimensions.

The normal cross-section interval shall be 100 ft (30m). However, in flat, low terrain longer intervals may be used and in rugged or mountainous terrain, 50 ft (15m) intervals may be necessary. Additional cross sections may be taken at odd intervals where significant changes in elevation are noted. Sections will be taken at all breaks in the centerline profile and at all other breaks in topography within the roadway prism and at cut to fill transitions.

It is not always necessary to put stakes in the ground at intersections of template and ground. For example, cross sections may be needed every ten or fifteen feet for quantity determination but the contractor needs stakes only every 50 ft (15m). Earthwork computations will compute all template-ground intersections between outside catch points. However, it is necessary that all groundline breaks and template breaks be recorded and that the outside slope catch points be found and recorded. The survey crew doing the staking should not deviate from this basic requirement.

Horizontal Curve An allowance should also be made for earthwork volumes around curves by decreasing the spacing of cross sections. This procedure will help compensate for inaccuracy of the average end area method (one contract documentation requirement) of computing earthwork quantities around curves. When encountering horizontal curves, as a recommendation, the cross-section interval should be:

- For curves with a radius of 5,000 ft (1500m) or greater, use a 100 ft (30m) interval.
- For curves with a radius of less than 5,000 ft (1500m) and greater than 300 ft (100m), use a 50 ft (15m) interval.
- For curves with a radius of less than 300 ft (100m) and greater than 150 ft (50m), use a 25 ft (10m) interval.
- For curves with a radius of less than 150 ft (50m), use a 10 ft (5m) interval.

It is common practice to take a cross section at each station interval; however, that is not necessary if other sections taken nearby more truly reflect the ground situation. Sections shall be taken at:

- PCs and PTs for simple curves
- TSs, SCs, CSs and STs for spiral curves
- the beginning and end of superelevation runoffs and runouts to curves
- drainage sites
- other pertinent locations that define changes in terrain

Vertical Curve It is recommended to use 50 ft (15m) intervals on vertical curves when the mid-ordinate between the curve and chord varies 0.2 ft (0.05m) or more between standard station intervals, 100 ft (30m).

10.7.2.3 Terrain Considerations

Construction volume computations may change significantly from the plan quantities unless cross sections are taken truly perpendicular to the centerline on tangents and to radius lines on curves. Where the right-of-way has been staked and lathed, sighting between centerline and the right-of-way stakes can ensure that cross sections are perpendicular. Right-angle prisms may be used to determine the perpendicular line on generally level terrain, but it may be necessary in rough or heavily timbered terrain to use a total station for right angles. In very rugged terrain, it may be necessary to run a series of offset lines parallel to the construction centerline before slope staking and cross sectioning to maintain right angles and to provide another line of elevation points.

As the depth of cut increases and virgin terrain is encountered, it is becoming a common practice in highway backslope design to build compound slopes. The staking practices now in use will give generally satisfactory results with only minor changes. It is suggested that only sufficient reference to cut depth be given to allow excavation down to the bench break, then reestablish another reference stake at or near the first bench or break to allow excavation down to the next bench, etc. This method of staking ensures that benches or breaks will be constructed correctly and has the advantage of simplicity. However, any other method that will provide proper slopes and place benches of the designed width in the correct location will be satisfactory.

When cross sectioning steep slopes, efficiency and speed can be aided by using reference hubs wherever possible. A reference line may also be used. Cross sections are taken right and left of construction centerline and extending to the right-of-way line or beyond the construction limits or construction permit as deemed necessary.

Notes for staking the grade on ramps or interchange areas should be made up ahead of time and should be sufficiently accurate to allow ready staking, but not so precise that weeks of time will be taken to make these notes.

Cross sections may be extended beyond the construction limits in situations where overbreak or slides are anticipated. Groundline readings should extend a reasonable distance beyond the outside grading stake.

10.7.2.4 Tolerances

The instrument height is to be recorded to the nearest 0.01 ft (0.005m), rod readings taken to the nearest 0.1 ft (0.05m) from a rod held vertically and measured to the nearest 0.1 ft (0.05m) horizontally.

10.7.2.5 Unusual Situations

Infrequently, the EPM may encounter an unusual problem or situation that does not fit the above procedures. When that happens, and quantities are to be determined by computer, the EPM should contact the appropriate Construction Bureau to reach an agreement as to procedures and methods to follow. Considerable time can be saved by determining the capabilities of the computer programs before the fieldwork is performed.

10.7.2.6 Slope Staking with Total Station

The availability of programmable calculators and coordinate listings makes using total station instruments a valuable option for slope staking, particularly in rough terrain. Some major advantages of using the total station are:

- no tape
- no deadman
- no “plus” rods
- no setup breaks
- no HIs or HSs

The total station readily produces more accurate horizontal distances than the tape. However, the vertical distances may not be as accurate unless the instrument is setup, leveled properly, vertically collimated and the prisms are accurately sighted.

There are various ways to use the total station for staking purposes, but the “Missing Line” is the most common used; thus, it will be the method described. For additional “Missing Line” information, refer to the user manual, as the process is explained in detail.

The “Missing Line” Method obtains data on an initial position and then displays the differences in slope or horizontal distances and vertical distances for each additional position. This can be accomplished without moving the instrument and without the instrument being online or in a potentially unsafe location. Note: subsequent positions need to have a prism with the same HS as the initial position, since no direct relationships to HIs and HSs are used.

As with other methods of slope staking, the template data needs to be calculated ahead of time to assure efficiency of the survey crew. Use the grades, notes, etc as if the more conventional “Rod & Level” Method was used. Establish existing centerline elevations using the same datum as the template and record it in the notes; therefore, the HIs are that same elevation for each station.

To begin the “Missing Line” Method of slope staking, setup the instrument in a convenient location that allows visibility to the area to be staked. Sight the prism on the initial position (centerline); this will record the initial position in the instrument. In the notes, write a zero vertical difference for centerline. Cross-section perpendicular to the centerline of the roadway assuring to pick up all major breaks.

For each additional position, sight the prism and start the “Missing Line” measurement. Once the information is displayed, record the horizontal & vertical distances in the notes. Note that a negative value for the vertical distance means that the new position is below the initial position. Continue this until you make the catch. Repeat for the opposite side of the roadway.

If the terrain allows, multiple stations can be slope staked per instrument setup; however, do not exceed the distance of 1000 ft (300m) from the instrument for any foresight.

10.7.3 Marking Slope Stakes

It is important that the information shown on construction stakes is concise, readable, and clearly understood by contractors. Because contractors may have projects in other parts of the State, consistency among the highway survey crews promotes understanding of the information being conveyed by everyone. Print the required information neatly on a painted stake and set them properly. Tilt them away from the centerline of survey at approximately 30 degrees from perpendicular. Face the printed side toward the centerline. On the backside, print the station identification. If a great deal of information is required, use two slope stakes.

Cut and fill stakes should be marked with the stationing, depth of cut or fill, the slope and distance to the hinge point (shoulder or ditch bottom).

It is recommended that the contractor be given a copy of the final staking notes. The notes show the location of the catches and the like, which makes the grade setter's job a lot easier.

10.7.3.1 Fill Stakes

Place fill stakes at the fill catch points and show the fill heights to the hinge point, the distances from the centerline of the roadway and the horizontal-to-vertical slope ratio (i.e. 4:1). Figure 10-1 and Figure 10-2 show examples of fill slope stakes.

Mark fill slope stakes to subgrade or safety shoulder (hinge point). In normal fill sections, these points may be different but will be called out in the plan Typicals.

10.7.3.2 Cut Slope Stakes

Place cut stakes at the cut catch points and show the cut heights to the hinge point, the distances from the centerline of the roadway and the horizontal-to-vertical slope ratio (i.e. 2:1). Figure 10-3 and Figure 10-4 show examples of cut slope stakes.

Mark cut slope stakes to the bottom of the ditch or to the back of the ditch (hinge point), if both elevations are the same. In normal cut sections, the bottom of ditch is the lowest elevation and the toe of the backslope.

10.7.3.3 Reference Stakes

Place reference stakes beyond all fill and cut catch point stakes and show the offset distances from the fill or cut stake, the fill or cut heights to the hinge point and the distances from the centerline of the roadway. Figure 10-5 shows example of reference stakes.

Mark reference stakes either to the subgrade or safety shoulder; to the hinge point; or to the respective stake it is referencing. Be consistent in the markings throughout the staking process.

10.7.3.4 Transitions

Stake shallow cuts and transitions from cut to fill with a "daylight" stake—marked "DL". This stake does not usually go out the full distance of the backslope cut. Stake it either level with the ditch bottom or lower than the ditch bottom. If you stake it level with the ditch bottom, mark it with "DL", F/0.0 to back of ditch, the distance from centerline, and the ditch distance from centerline. Figure 10-6 shows example of daylight stakes.

10.7.3.5 00 Sections

Place 00 stakes at locations where the ditch or shoulder would be extended beyond the normal distance from the hinge point at the same slope rather than placing an actual fill or cut section at that station. Show 00 “DT” or 00 “SH”, respectively, the distances from the centerline of the roadway and the horizontal-to-vertical slope ratio (i.e. 4:1) or percent grade (2%). Figure 10-7 and Figure 10-8 show examples of cut 00 sections.

10.7.3.6 Culvert Stakes

Place “EP” stakes at the end of culverts and show the fill or cut heights to the culvert flow line, the distances from the centerline of the roadway and the station. Place an “EP” reference (“RP”) stake at a convenient offset distance from the “EP” stake and show offset distance to the “EP” stake, the fill or cut heights to the culvert flow line, the distances from the centerline of the roadway and the station. Figure 10-9 and Figure 10-10 show examples of culvert and culvert reference stakes.

Mark “EP” and reference “EP” stakes to the flowline of the culvert. Further, mark the stake with size and type of the culvert.

10.7.3.7 Reconstruction

Many highway projects are reconstructed on existing alignment. Much of the staking of cuts, fills, and appurtenances can be done as described above. Other cases, however, present unique problems, particularly for the location and marking of construction control staking.

10.7.4 ROAD APPROACHES, DIKES, ETC.

Most road approaches and small dikes will be constructed from a few guide and reference stakes. Where it is necessary to stake an approach or dike, treat it as a separate alignment using centerline, shoulders, and slope stakes. The quantities may be included in the roadway mass.

10.7.5 STAKING BORROW PITS

Frequently, borrow material is used to maintain a more uniform grade line and to construct design features in locations where roadway excavation material is not readily available. One of the primary considerations in removing material from a borrow site is determining how much material has been removed for roadway construction and where it was placed. Other considerations involving borrow sites are: topsoil removal and replacement, proper drainage, and appearance after the borrow material has been removed.

Borrow pits can be measured by cross sections, Digital Terrain Models (DTMs) or other methods approved by the Department. It is recommended to use the DTM Method of measuring borrow pits as it is much easier to establish the datums and to do periodic measurements.

When measured utilizing conventional methods, the level datum for borrow sites is to be tied in with the roadway bench marks where possible. Where borrow pits are located adjacent to the roadway, the roadway cross sections may be extended for complete coverage of the borrow site. Where it is not practical to tie the borrow site elevations in with the roadway level datum, at least two bench marks should be established outside the construction operations and referenced to an assumed elevation.

The borrow site notes should contain a sketch of the borrow location including fences, baseline stationing, witnesses, references, bench marks, buildings, borrow limits, north arrow, a description of the section, township, range and other pertinent information that would allow another person to reestablish or carry on the work.

Topsoil is to be removed to the required depth, stockpiled and replaced after the borrow material has been excavated and the borrow area reclaimed. The topsoil quantities are to be kept separate from other excavated materials. This may require cross sectioning and computing before the topsoil material is removed, after the topsoil is removed, and again after all the borrow material has been removed. In any case, the excavated quantities and the topsoil quantities need to be kept separate.

Examples of different methods of staking borrow sites may be found in Figure 10-11.

10.8 FINISH GRADE STAKES

10.8.1 GENERAL

The operation of construction staking is not complete until the roadway; ditches and other related facilities have been completed to the design section. Finishing stakes, called bluetops, are set by the contractor, unless otherwise specified in the contract documents, to line and grade as the roadway progresses from rough to finish grade. The roadway centerline is set and the shoulder stakes are measured perpendicular to centerline. The EPM will check bluetops in accordance with the contract documentation.

Finish grade stakes shall be set in accordance with contract documentation. Elevations are established using a level or other method approved by MDT.

10.8.2 GRADE CONNECTIONS TO EXISTING

Care should be exercised to provide a smooth transition where the roadway grade meets an existing bridge or other changes in elevation occur. Verify the elevations shown on the plans match the actual elevation at all tie-in points. Changes in elevation should be extended to make the transitions as gentle as possible.

10.8.3 CONCRETE PAVING

Grade and line for concrete paving shall be in accordance with the contract documentation.

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10.9 BRIDGE CONSTRUCTION SURVEYS

10.9.1 General

The bridge staking method is designed to provide an independent check of all structure layouts. This check will benefit both the contractor and the Department in that delays and expenses incurred due to improper location of structure units will be held to a minimum. This method will also aid in maintaining engineering costs at acceptable levels. Bridge construction surveys are to be completed in accordance with contract documentation.

10.9.2 Bridge Surveys

Layout and engineering control of bridges requires precision not necessary or generally practiced on most other highway construction. This is due mainly to the materials used in the construction of bridges. Once concrete is in the forms and hardened, correction of any mistakes is very costly and time consuming. Errors in span lengths, column lengths, cap elevations, etc. generally require costly corrections. For these reasons, it is necessary that every effort be made to provide this precision.

On projects with multiple structures, it is desirable to allow sufficient time for establishing primary controls for all structures because this permits establishing the controls with a minimum of time and effort and frees personnel for inspection duties during construction.

Equipment capable of producing the required accuracy shall be made available and used. The precision required for bridge construction survey controls justifies the use of more sophisticated equipment than may typically be used on highway staking.

10.9.3 Checking of Work

All project personnel should be aware of the benefits of checking construction layouts. Rough checks that take very little time or effort—visual comparison of column heights to approach fills and other bents; and taping between like units on separate bents—can disclose gross errors in design or layout and prevent costly extra work. The elimination of errors is mainly dependent on constant checking.

10.9.3.1 Responsibilities

Bridge construction staking shall be completed in accordance with the contract documentation. Typically, the contractor is responsible for the completion of bridge layout.

10.9.4 PRELIMINARY PROCEDURE

Before construction work may begin on any structure, layout referencing and field checking of primary horizontal and vertical control shall be complete. Reviewing of bridge layout should include, but is not limited to the following:

- Check dimensions on bridge plans prior to staking. In particular, check elevations from finished grade down through the deck slab, girder, shoes, crossbeams, columns and footings. Check span lengths, skew angles, and horizontal control.
- Review road or other plans for construction features that could affect the layout and location of references for the structure—especially any special embankment requirements.
- Check alignment and stationing of roadway centerline as staked in the field. Be certain that lines and stations staked on the ground are as shown on the plans.
- Determine whether bench marks shown on the plans and those proposed for establishment of vertical control are in place and usable.

10.9.5 STATE PLANE COORDINATE CONSIDERATIONS

Because of the details of bridge design and construction, the plans are designed and produced using a combination of ground and grid dimensions.

Stations shown on the bridge plans are state plane grid stations based on state plane coordinates. Dimensions shown on the bridge plans are horizontal ground distances and not state plane grid distances. The bridge plans should indicate the horizontal and vertical datum as well as the combination scale factor (CSF).

10.10 CONSTRUCTION SURVEYS FOR DRAINAGE FACILITIES

10.10.1 GENERAL

The proposed flow line elevations for outlet ditches, sewers, channel changes, culverts and other drainage facilities are shown on the plans. However, during the time interval from plan completion to contract award, changes may have occurred. Therefore, culverts and other drainage facilities, as planned, may need to be revised to fit existing conditions. It is a good idea to make a thorough review of the existing drainage course compared with the location shown on the plans.

Existing culvert or storm drain flow lines through adjacent highways, railways or other facilities may effectively control flow of water through the intended construction zones. Drainage facilities through the new roadbed may require adjustment to conform to in-place drainage facilities.

10.10.2 CULVERTS

Culverts may be staked and installed before grading begins, and the staked length must be known so the contractor can order accurate quantities. Although the location, type, size, length and elevation of the flow line are shown on the plans, all conditions affecting these requirements should be checked before the culvert is staked. Culverts should be staked and the staking information made sufficiently clear so there will be no error on the part of the contractor or inspector due to misunderstanding. Skew angles, as shown on the plans, should correspond with field conditions, but if they do not, corrections shall be made so the installation will conform to field conditions.

The staked length of culvert pipe shall fit the slope, and generally should conform to the commercial lengths available for that size of pipe. An end of pipe (EP) stake is placed at the end of each culvert stating the centerline offset. A hub is set at each end of the pipe centerline a sufficient distance beyond the construction limits to prevent their destruction. A guard stake placed over each hub should show the cut or fill to the flow line and the distance offset from the end of the culvert. Other information relative to the length, type and size of pipe needed for the installation may also be shown on the guard stakes or a lath.

After the stakes have been set, a check should be made of the total fall and direction of flow. A gradient of 0.3 percent for reinforced concrete pipe (RCP) and 0.5 percent for corrugated metal pipe (CMP) are the minimum grades that will carry sediment.

Unless elevations are specified in the plans, culverts should be placed with a gradient equal to or greater than that stated above for the type of pipe being installed.

When invert elevations are not specified in the plans, the culvert invert should be staked about 10 percent of the pipe diameter lower than the flow line elevation of the inlet and outlet channels to help minimize piping, obtain a more solid bedding, and provide more opening for small flows.

10.10.2.1 Concrete Box Culverts

Staking box culverts should include but is not limited to the following:

- Review and check plans and shop drawings for concrete box culverts prior to stakeout.
- Set reference lines parallel to the centerline of the structure at each edge or at an offset to the edge. The reference lines may be used to set alignment and screed rails for installation.
- Place reference stakes outside the construction area at each end of the structure centerline and at convenient locations on the other reference lines.

The need for reference lines is determined by the EPM, but is based on the width, length and location of the culvert and should accommodate the manufacture's installation recommendations. The EPM should explain the staking method to the contractor and the inspector.

10.10.3 DIKES AND DITCH BLOCKS

Stakes marking the extremities of the dikes are usually satisfactory. However, an adequate number of stakes should be set to delineate all of the features of the dikes and to allow an accurate computation of earthwork volumes involved.

Staking of ditch blocks can be done by setting a stake on the shoulder. This stake will then designate the stationing of the ditch block and its height, length and side slopes. The dimensions of the ditch block should be set in accordance with contract documents.

10.10.4 INLETS

A single stake, hub or marker may be used to identify the center of the inlet structure, and two or more reference stakes may be used to establish the structure centerline by marking the distance from the center of the structure relative to the references. Inlets frequently meet existing curbs or other in-place structures; therefore, shall be accurately staked to avoid alterations.

10.10.5 Curb, Gutter and Sidewalks

A reference line will be set behind the curb at a constant offset distance and referenced with hubs and tacks. The EPM should discuss offset distances with the contractor prior to running the reference line. Offset distances should be marked to the top back of curb. An adequate number of references shall be placed on the reference line to insure proper grade and alignment. The horizontal and vertical controlling points should be included in the reference line. It is also recommended to include the radius point if possible.

The vertical difference in the elevation between the reference point and the elevation of the top back of the curb should be included. Existing grade control points such as intersecting streets; in-place sidewalks, inlets or in-place curb and gutter should be used to modify the planned elevations.

Sidewalk is typically built in relationship to the top back of curb.

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10.11 CONSTRUCTION SURVEYS FOR MISCELLANEOUS ITEMS

10.11.1 GENERAL

Items in this group include fences, bin or retaining walls, signs, pavement markings, utility items, guardrail, and comfort stations.

The number and degree of accuracy used in setting construction stakes for miscellaneous items should be consistent with the use of that item and should be adequate to establish the location.

10.11.2 FENCE

The latest right-of-way plans and agreements are to be made available to the survey crew for use at the time fences are staked. An adequate number of stakes should be set at reasonable spacing to allow fence construction without extensive use of surveying instruments. The survey crew should recognize that staking of fencing along any line shown as right-of-way equals existing on plans is only approximate. If it is critical to know the exact location of the existing right-of-way, a retracement survey of the existing right-of-way is required.

10.11.3 BIN AND RETAINING WALLS

A satisfactory number of cross sections should be taken from a baseline to the bottom of the bin or retaining wall to calculate volumes. The cut or fill from this baseline may then be calculated to other control points of the bin or retaining wall. Establish reference lines and set control points outside the construction zone to readily reestablish points along the wall.

10.11.4 SIGNS

Stake signs at the locations shown on the plans with a lath, stake or hub. The sign is identified on the lath or stake. Signs may be moved from the planned position where obstacles are encountered on the approval of the appropriate authority. Stake signs in accordance with the contract documents.

10.11.5 PAVEMENT MARKINGS

Stake pavement markings at locations shown on the plans under the direction of the EPM. Stake beginning and ending of tapers, lane lines, cross walks, stop bars, hatching, etc. Stake locations of symbols as appropriate. Stake all pavement markings in accordance with contract documents.

10.11.6 UTILITY RELOCATION

Most utility moves, relocations or adjustments are made by the utility owners prior to construction, which generally limits the staking required to that necessary for establishing horizontal and vertical controls.

Utilities that are being installed relocated or readjusted, as a part of the contracted work will require staking necessary to provide alignment and grade in addition to horizontal and vertical clearance controls. A careful review should be made of the proposed roadway construction so adequate clearance is provided across road approaches and the main line, so the intended position of the utility does not conflict with use of other facilities.

10.11.7 GUARDRAIL

Installation instructions are shown in contract documents. Adjustments may be necessary in plan lengths and locations. This may require proper authorization before proceeding with staking.

Different staking methods may be used. The method used will be determined by the EPM and explained to the inspector and the contractor.

10.11.8 COMFORT STATIONS

Comfort stations require staking building locations and such items as drain fields, underground sprinkling systems, driveways, sidewalks, and locations for planting trees and shrubs.

A careful review of the plans for the building site and attendant facilities should be made before staking is begun. Baselines or reference lines should be established at the proper locations and their control points referenced to prevent destruction. Line and grade shall be set for the footings of the building, for sewer facilities, and to provide a convenient reference for workers to establish batter boards and lay out the facilities.

The following figures are intended to illustrate the proper procedure for marking stakes and reference stakes. In addition to stakes, hubs should be utilized where necessary. For clarity purposes, the locations of the hubs are not shown in the figures.

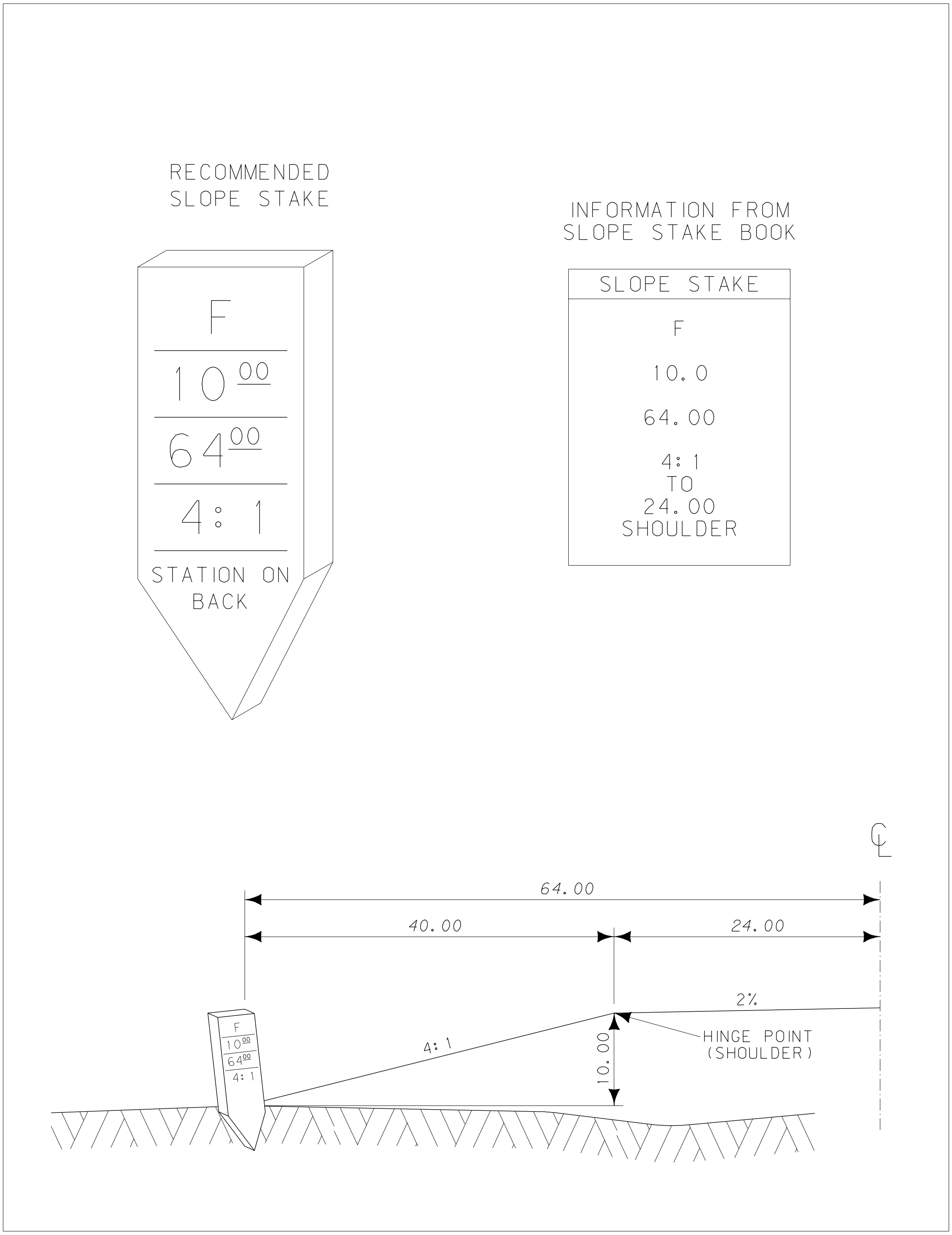


Figure 10-1
Slope Stake for Normal Fill Section

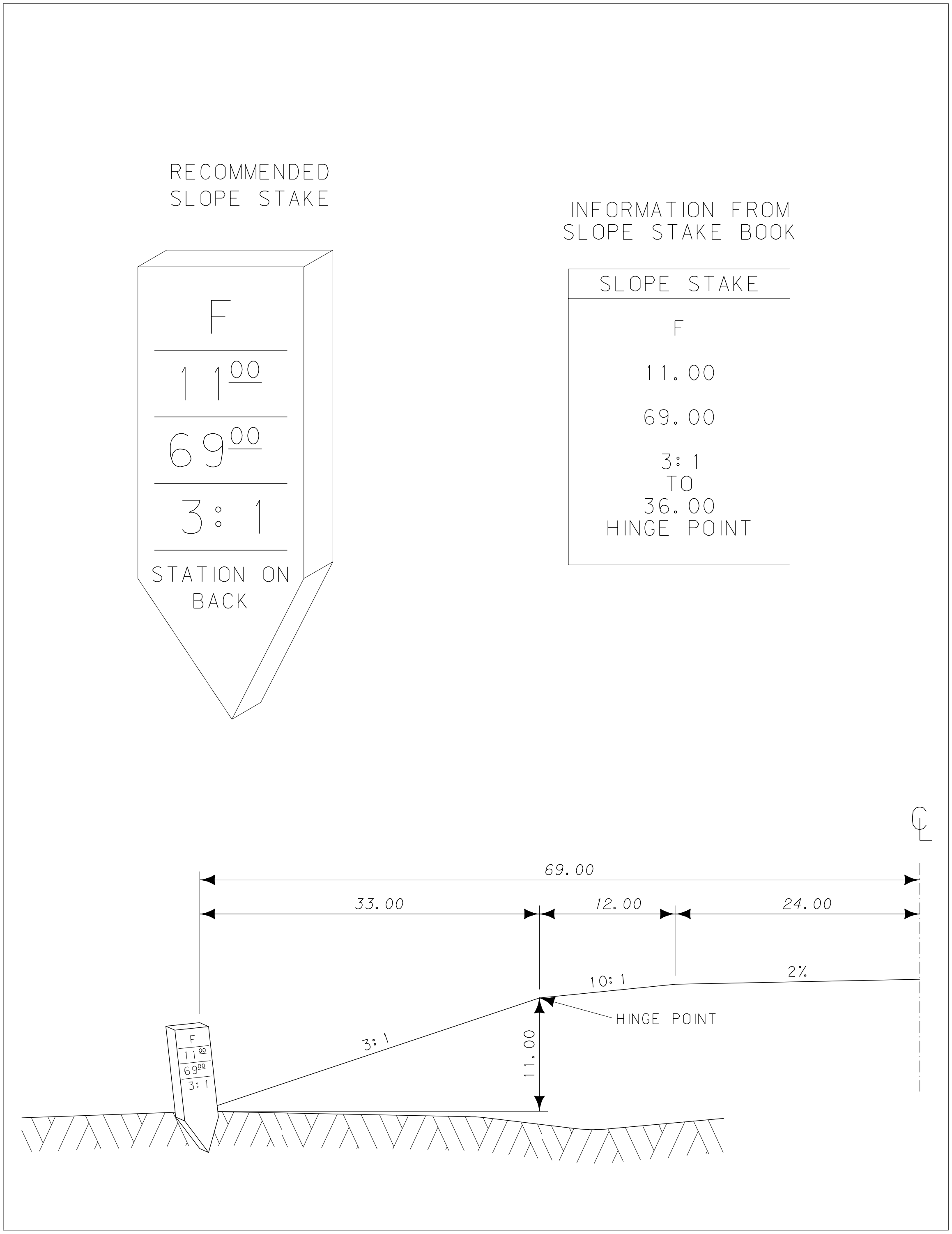


Figure 10-2
Slope Stake for Fill Section with Safety Shoulder

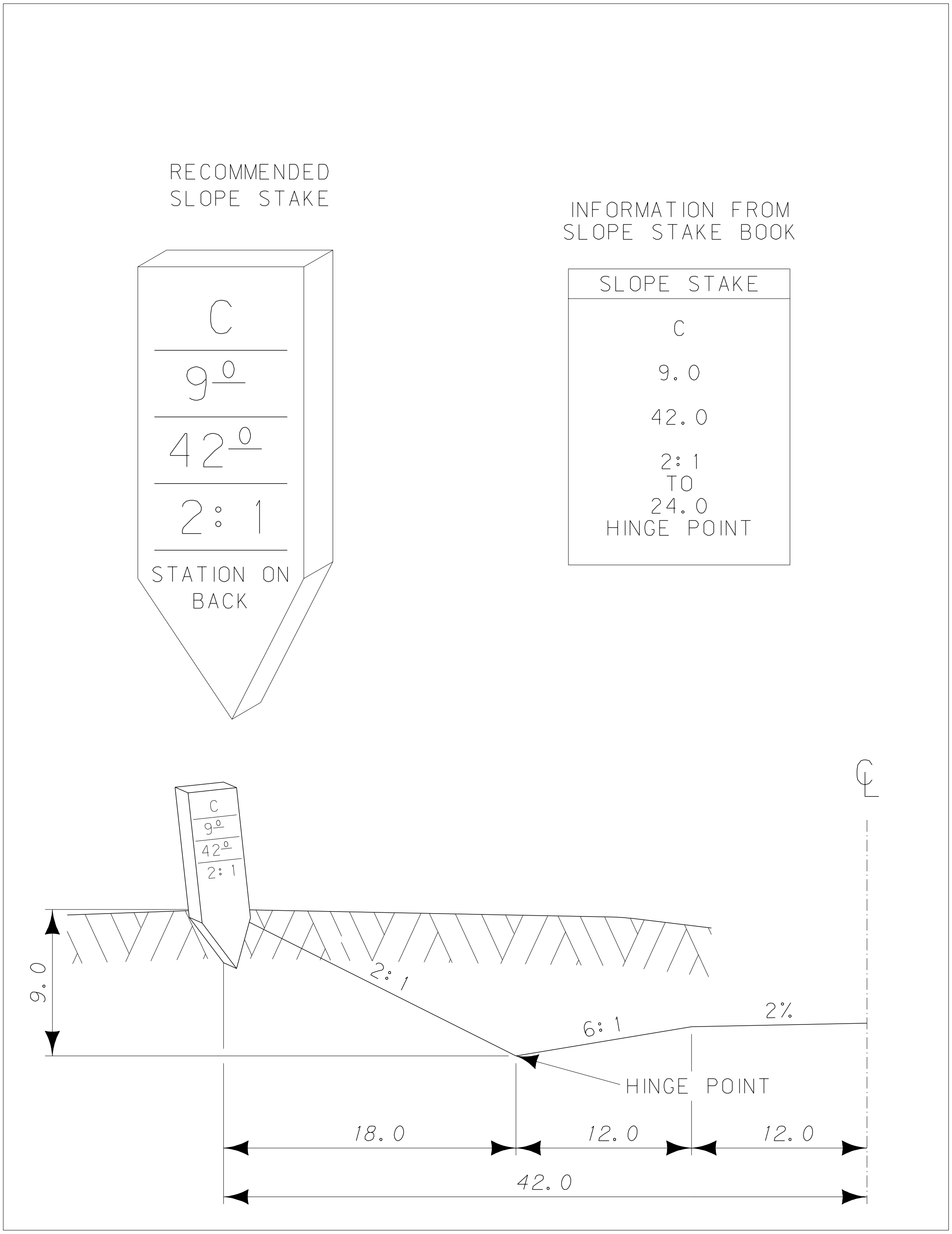


Figure 10-3
Slope Stake for Cut Section—V-Ditch

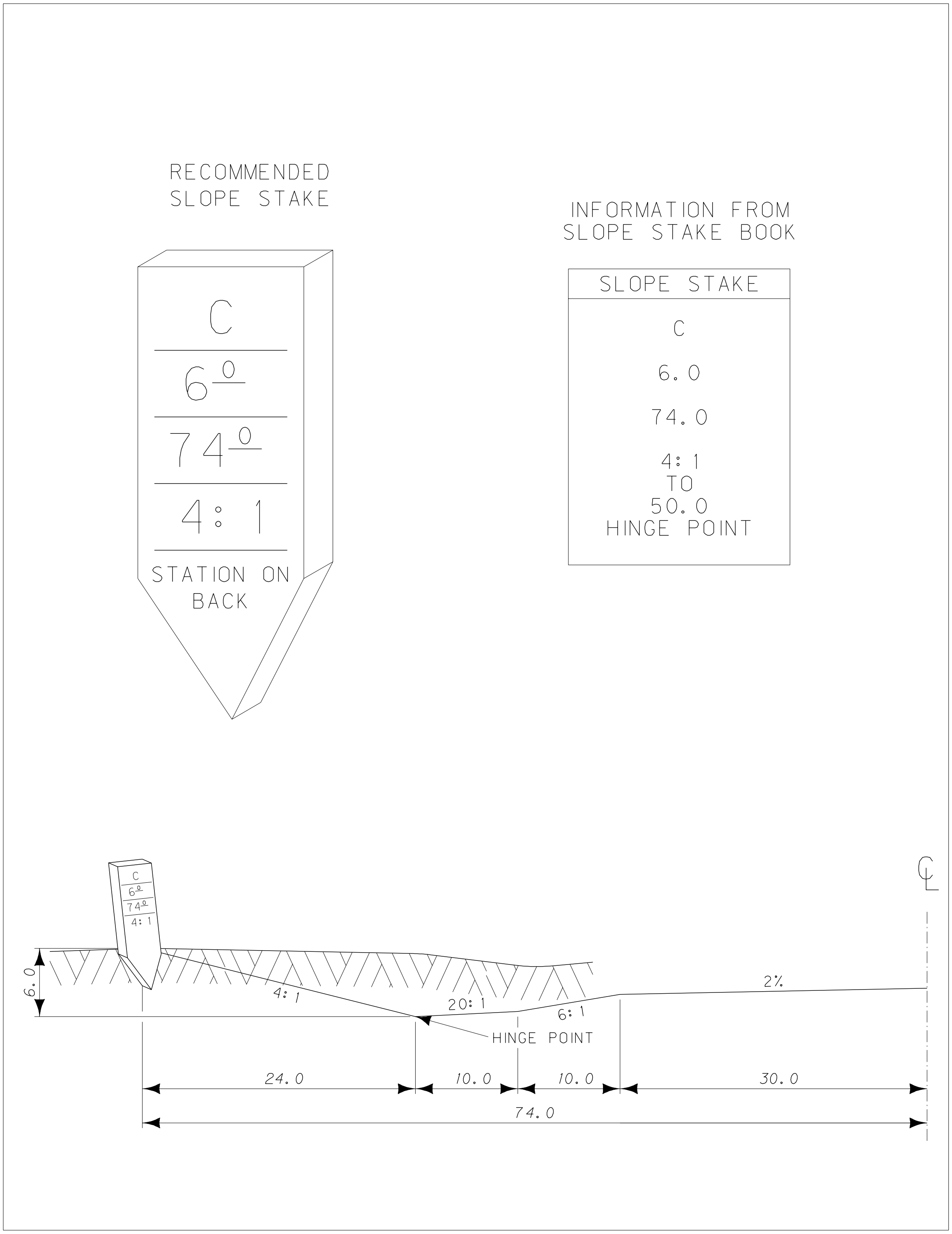


Figure 10-4
Slope Stake for Cut Section—Flat-bottom Ditch

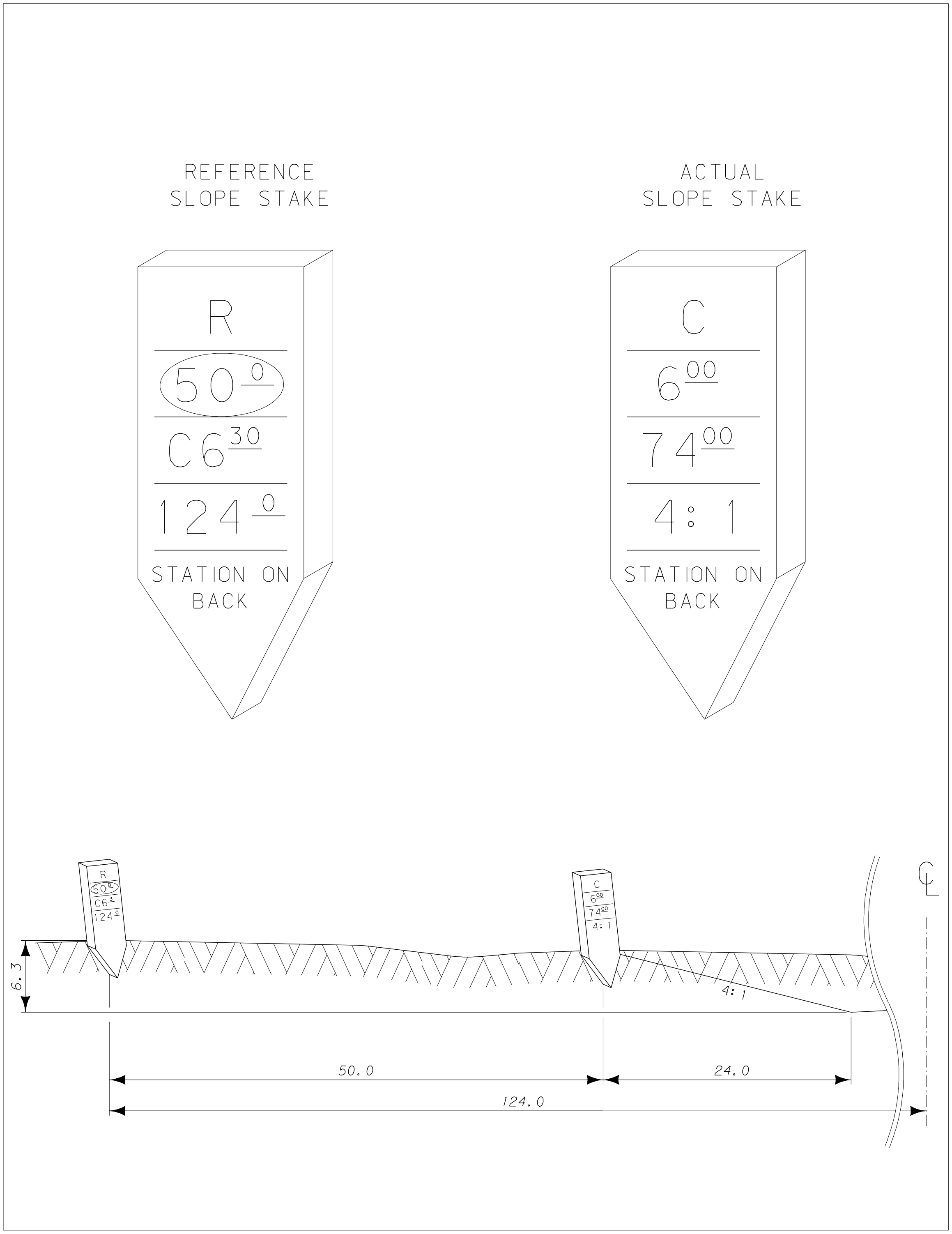


Figure 10-5
Reference Stake for Slope Stake

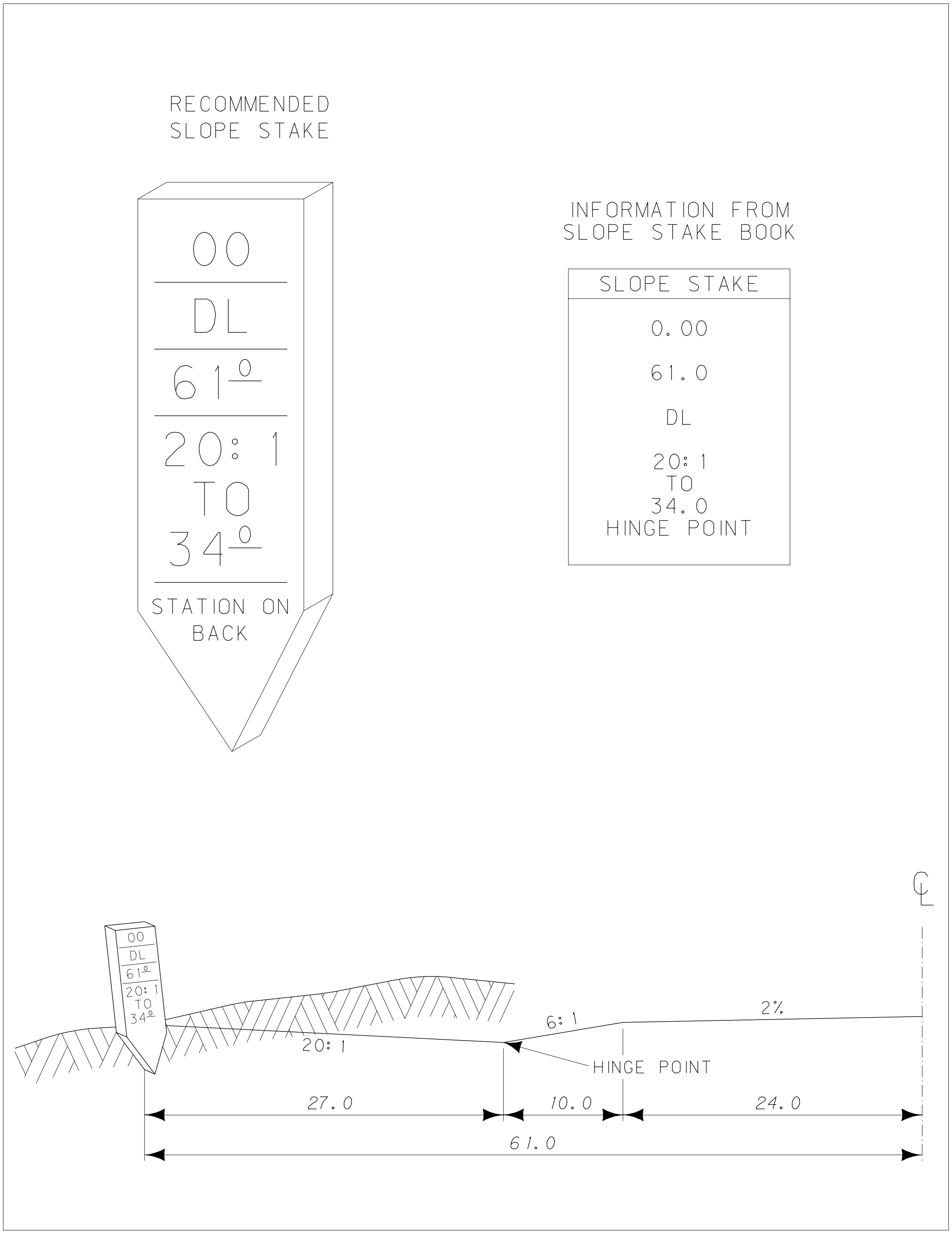


Figure 10-6
Slope Stake for Daylight Section

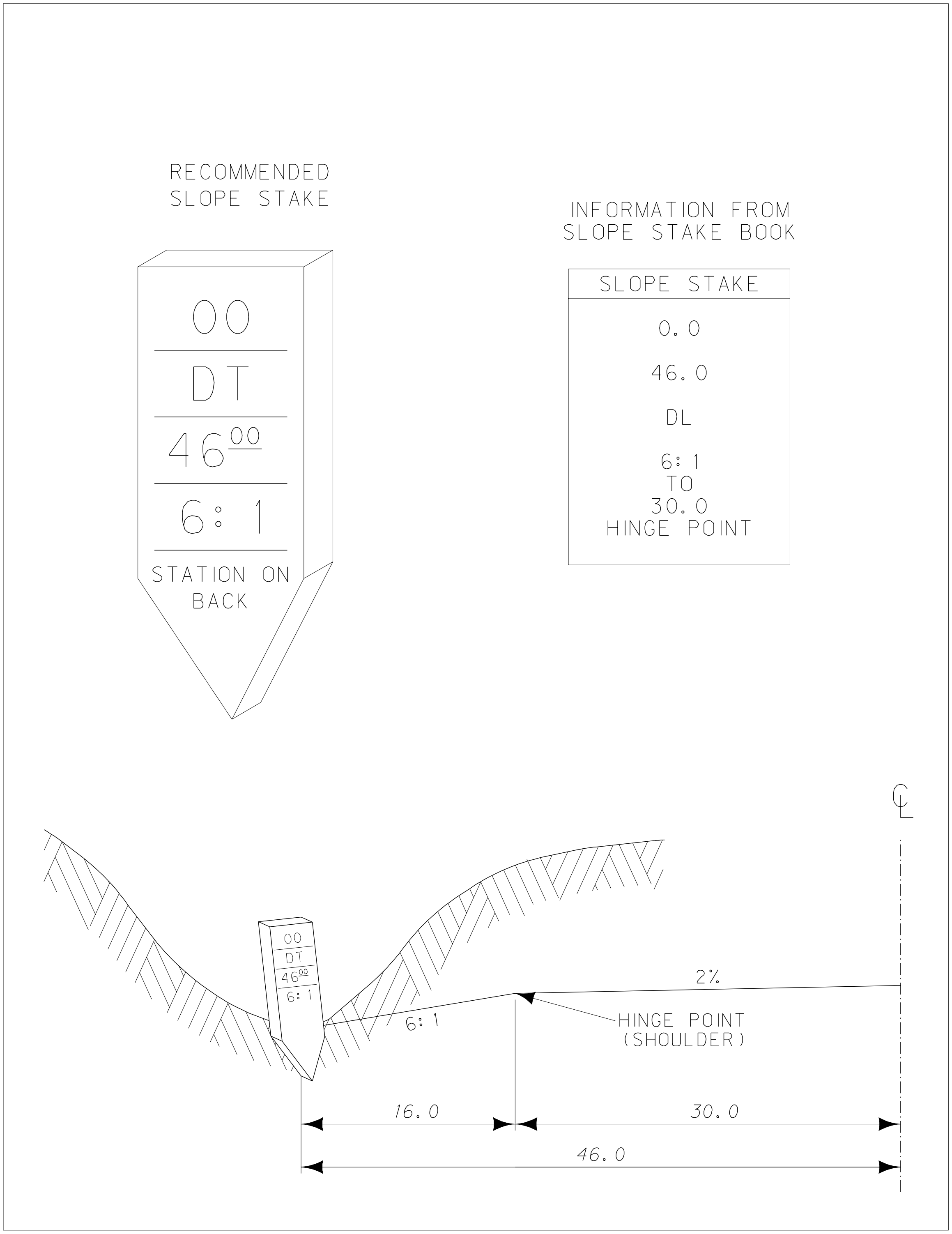


Figure 10-7
Slope Stake for 00 Ditch Section

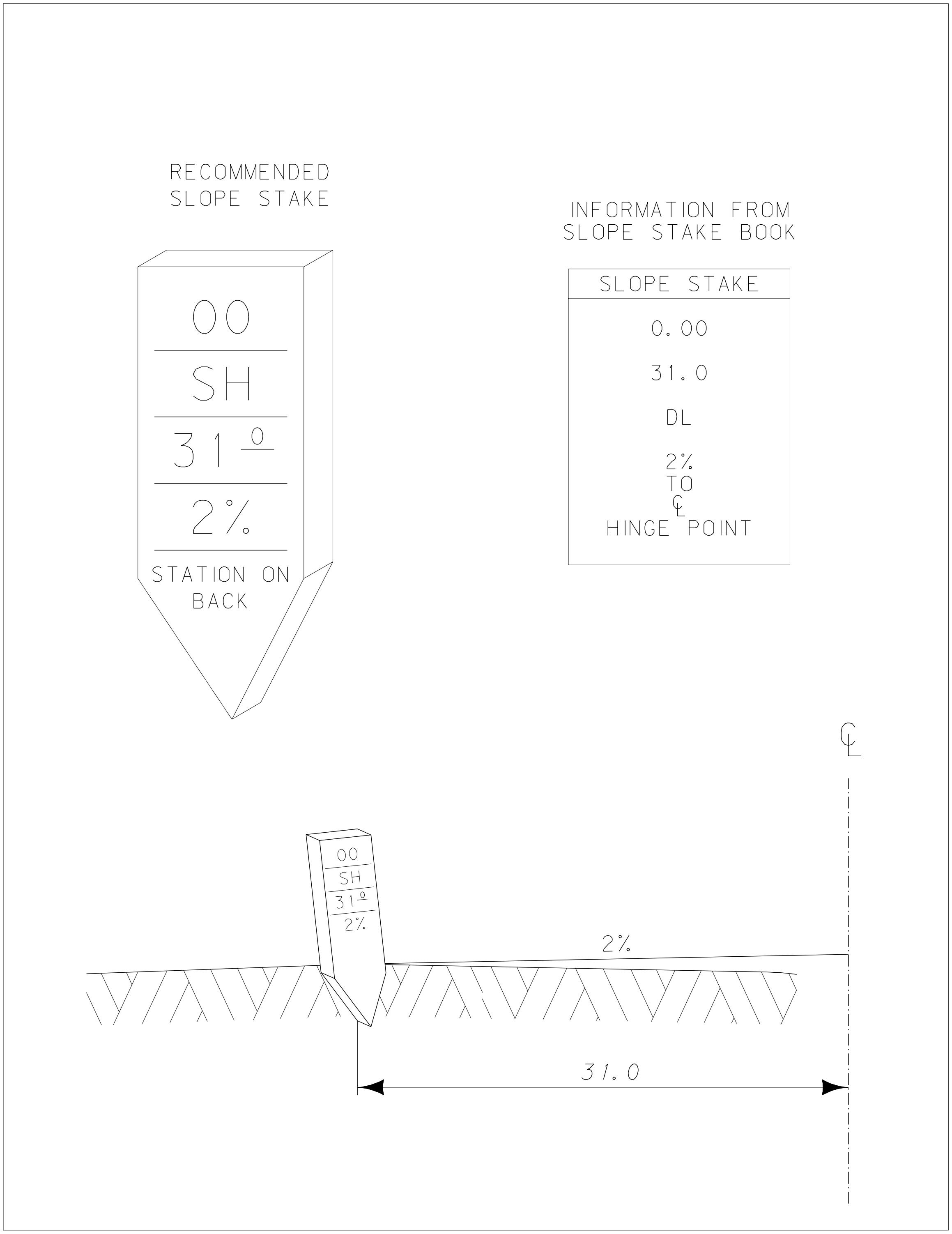


Figure 10-8
Slope Stake for 00 Shoulder Section

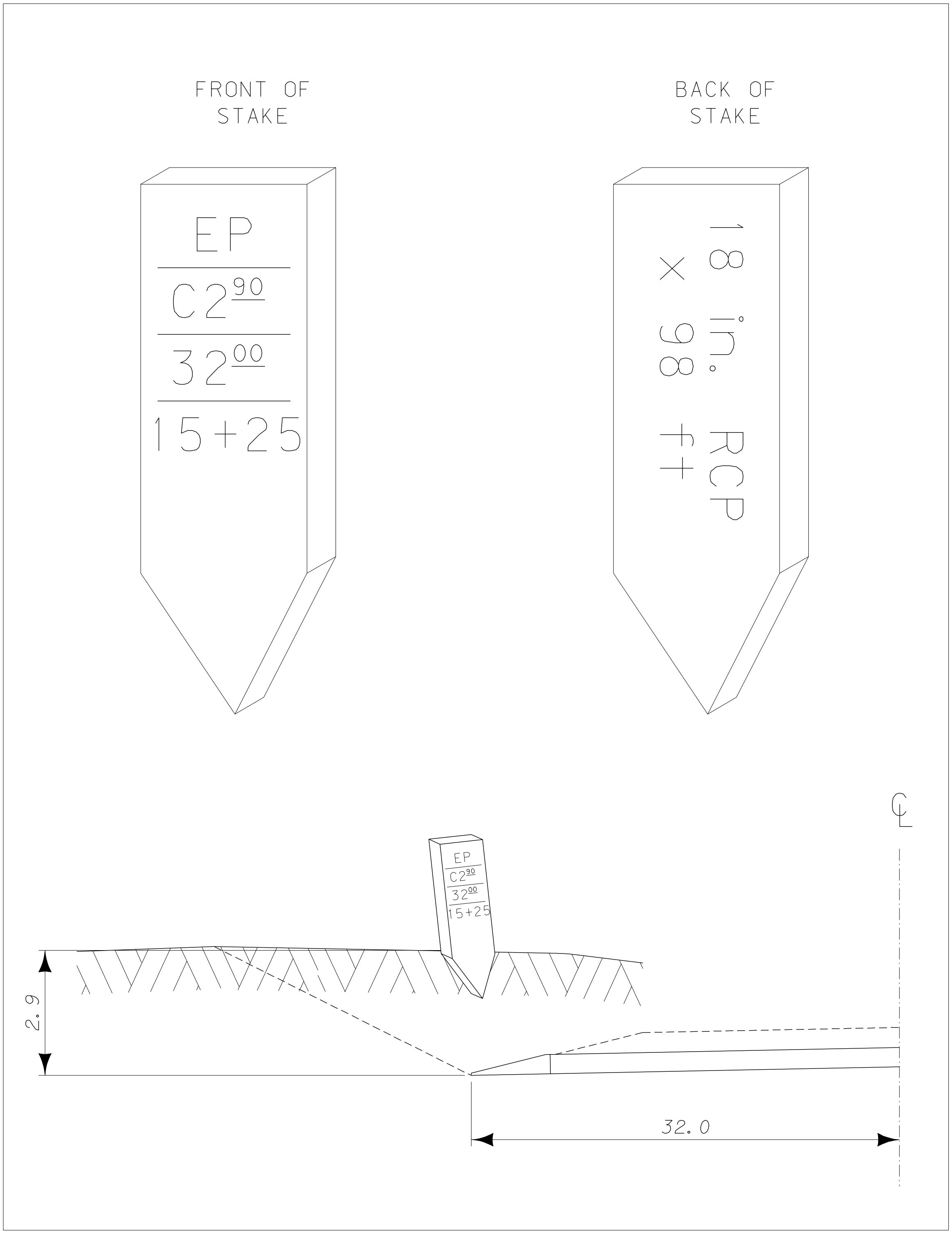


Figure 10-9
Slope Stake for Normal Culvert Section

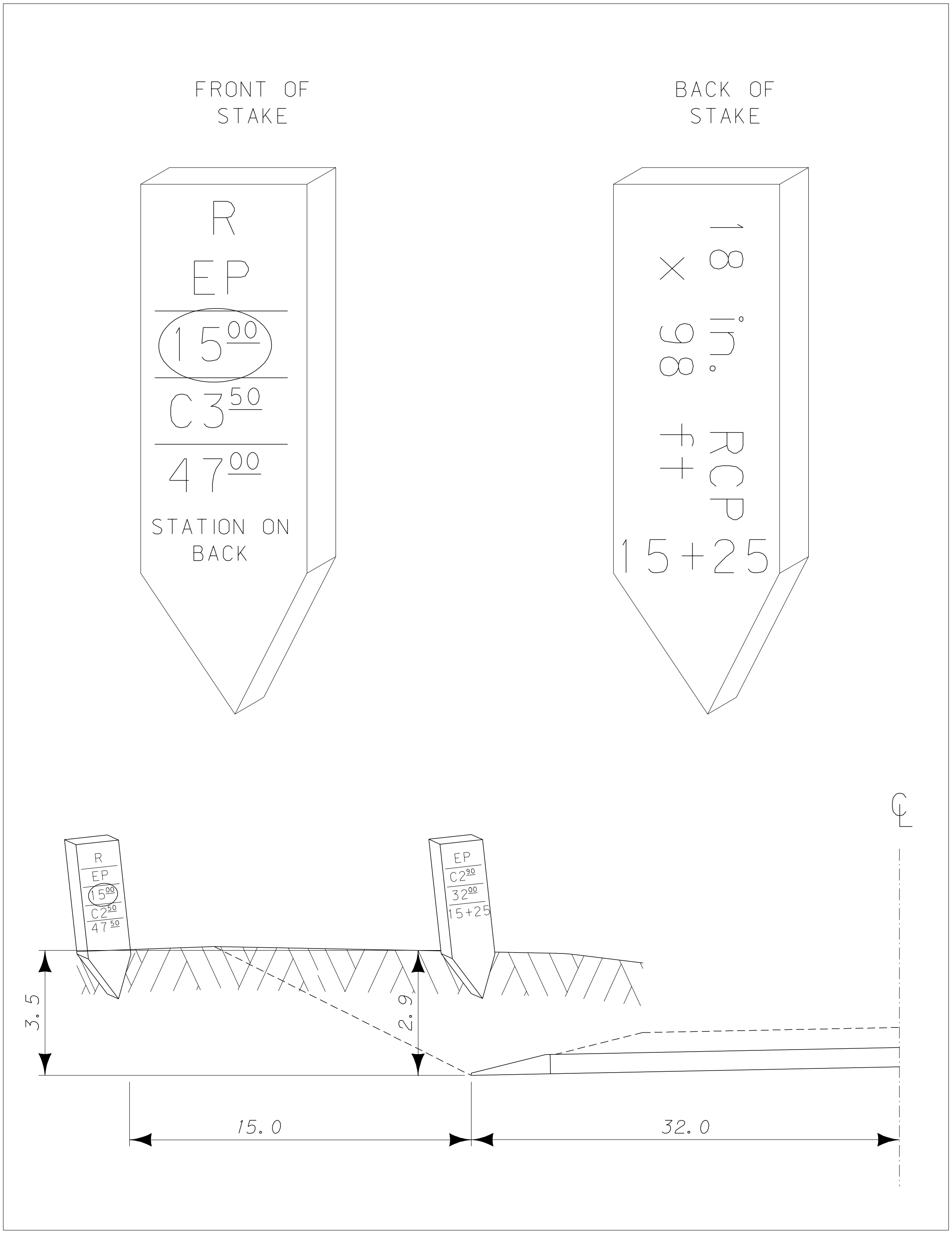
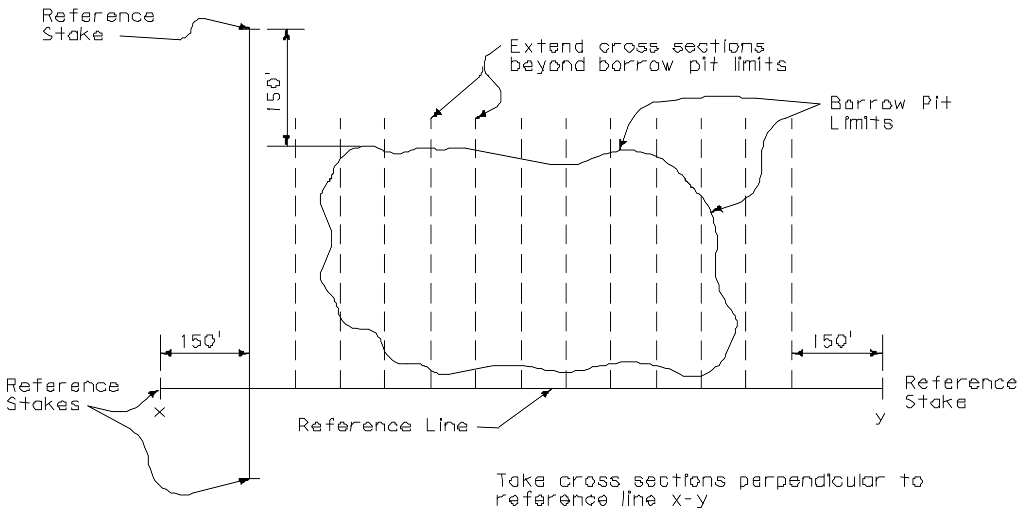


Figure 10-10
Reference Stake for Culvert Section

Take cross sections at each intersection of lines forming a 25 foot grid network. Use other points, as necessary, to show the true ground surface. Disances or dimensions shown below are for the surveyors guidance. Use judgement to fit the survey to the natural terrain.

1. Typical Borrow Excavation Site



2. Borrow from Old Embankments or Long Rows of Material

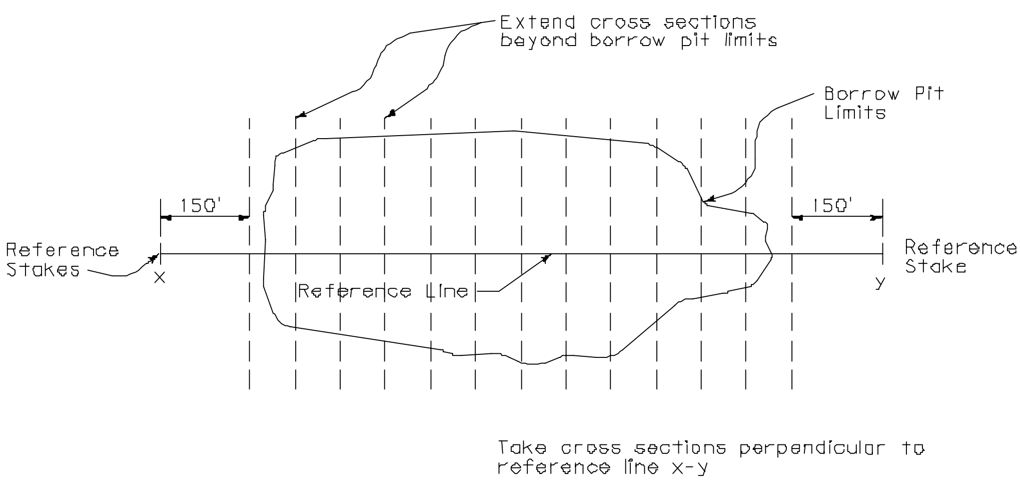


Figure 10-11
Borrow Pit Cross Sections

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Hydraulic Surveys

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Chapter 11
Hydraulic Surveys

This chapter describes the general survey requirements for the Design of Waterways, Irrigation Surveys, Urban Storm Drains, Culvert Surveys, Small Earthen Dams, and Roadside Erosion. Site-specific information is included in either the Location Hydraulic Study Report (LHSR), or the Preliminary Field Report (PFR). Survey requirements found in the LHSR or the PFR supersede and/or supplements the general survey requirements found in this chapter.

All hydraulic surveys are different and frequently items will be encountered that are not discussed in this chapter, the LHSR, or the PFR. Contact the District Hydraulic Engineer to discuss the information that should be submitted for a particular hydraulic site.

The old saying, “A picture is worth a thousand words,” is especially true when discussing a hydraulic survey. The need for adequate photographs cannot be over emphasized. A great deal of information can be submitted in photographs. Label digital photos using a photo editor program and include a digital photo index in spreadsheet form with each hydraulic survey.

11.1 SURVEY DATA FOR DESIGN OF WATERWAYS (HYD-1)

Advancements in data collection created a need for some changes in the procedures outlined in this section. All the previously obtained information must be included in the survey as before, but some of the information will be in a different format.

An updated HYD-1 form has been developed to replace Sections 1 through 4 of the old 1993 HYD-1. The survey crews are required to complete Sections 1 through 3 of this new form. See Form 11-1 for an example of a completed HYD-1 located at the end of this chapter. The most current HYD-1 form can be downloaded at:

http://mdtinfo/survey/net/internal/documents/dcs_form_hyd-1.doc

Survey crews are still required to collect all pertinent data with the data collector and submit all data in electronic format. Prior to final submittal, the survey crew must attempt to coordinate with the Road Designer to review the XXXXDIMAP001.DGN file that includes the hydraulic survey data for submittal to the Hydraulics Section. Hydraulics and/or Road Design will further render the hydraulic survey information into a usable format for the Hydraulics Section. Consultants shall adhere to the terms of their contract with the Department for deliverable requirements. Refer to Chapter 1 for additional information regarding Consultant Surveys.

Complete Form HYD-1 for all waterways and crossings as designated by the LHSR or PFR. Form HYD-1 should be filled out electronically and submitted with the hydraulic survey data. All supporting electronic documents, such as the readme file, road design map file, xyz file, digital photographs, etc., are submitted electronically with the Form HYD-1. All bench level notes must indicate the datum, e.g., NGVD29 or NAVD88. If the survey is completed with the NAVD88 datum, the vertical difference between NAVD88 and NGVD29 will be provided as part of the final control survey. All digital photos are labeled and dated using a photo editor program; and a digital photo index is included in a separate spreadsheet file. Some of the information requested will not be readily available and will require some research or consultation.

The following subsections describe in detail the required data for the HYD-1 form. The data for each item required on the form is placed on the appropriate blank completely and accurately.

11.1.1 WATER CROSSING INFORMATION (HYD-1, SECTION 1)

This section contains information about describing water crossings in terms that will be usable by the engineer. Other items such as debris and ice are included because they have a direct bearing on the type of structure to be selected. See Form 11-1 for a completed HYD-1 Water Crossing Information example.

11.1.1.1 Floating Debris

Flood flow reaching a culvert or bridge nearly always carries debris that may be either floating material, material heavier than water, or a combination of both. Debris must be considered in the hydraulic design because it can be deposited at the culvert or bridge entrance and impair its operation. A thorough study of the extent and types of the debris originating in the drainage basin is essential for proper design.

The survey crew needs to make an observation at the time of the survey on the amount and size of the existing debris. This additional information is valuable to the design since the surveys are typically completed at different times of year than the Preliminary Field Review.

The quantity of floating debris has been classified into the following:

- none - very light or no debris
- light - small limbs or sticks, orchard pruning, and rubbish
- medium - limbs or large sticks
- heavy - logs or trees
- other - city trash, old car bodies, etc.
-

The group that best exemplifies the debris from the drainage basin should be indicated on Form HYD-1. If the “other” classification is checked, a short description of the kinds of debris found should be attached. Possible future changes in the type of debris that might result from changes in land use in the drainage basin should also be noted. As an example, logging in a previously virgin area could change the nature of the debris problem from one of “medium debris” to “heavy debris”.

The engineer needs an estimate of the quantity, as well as the type of debris, so that adequate debris storage can be provided. Information on the types and quantities of debris resulting from past floods can be an invaluable guide in selection of the type of debris control structure. Such information may be obtained from maintenance personnel, local residents, or personal observation.

11.1.1.2 Ice

Ice conditions, like debris, have a direct bearing on the type of structure and substructure used. Ice can cause damage at hydraulic structures in two ways: (1) ice can exert high forces or pressures causing a structure to buckle or collapse, either by thermal expansion during freezing or by impact of large chunks of ice on the structure at high velocities; and (2) ice can plug the waterway causing flooding to upstream property and possible damage to the structure.

The potential for ice damage at a crossing should be determined and indicated on Form HYD-1 as either:

- light [0 to 0.5 ft (0 to 150mm) thick]
- moderate [0.5 to 1 ft (150 to 300 mm) thick]
- severe [1 to 2 ft (300 to 600mm) thick]

If it is determined that the ice damage could be severe, a discussion of the type of damage and how it could be caused should be attached. Maintenance personnel and local residents may aid in establishing whether or not there is a potential ice problem.

11.1.1.3 Water Surface Elevations

Water surface elevations should be collected at the current edge of water at 25 ft (10m) intervals. Water surface elevations should be collected on the right and left edge of the active stream and should extend 25 ft (10m) beyond the furthest upstream (500 ft ≈ 150m) and downstream (1500 ft ≈ 450m) cross sections.

Water surface elevations can fluctuate over the period of the survey; therefore, it is necessary to note each date water surface elevations are collected for the different stretches of the waterway. This information shall be recorded in the readme file by including the upstream or down stream distance and date collected. Also, record the normal water surface elevation (vegetation / bench line) at the centerline of the structure on the bank of the channel.

The high water elevation requested on Form HYD-1 is the historic high water elevation. This elevation can be established in several ways. There may be visual signs such as drift lines or water stains immediately upstream of the crossing that will indicate the high water mark. Water stain locations may also be collected on the roadway at locations where roadway overtopping has occurred. Local residents, Department maintenance personnel, county commissioners, and county road foremen (on secondary projects) will many times remember how high the water got at certain points and may help establish the high water elevation. The dates and causes of high water should be given if they can be determined. Finally, record any historical incidents and frequency of structure or roadway overtopping events witnessed by locals, maintenance, or landowners.

Establishing the location, nature, and elevation at which damage would occur from upstream flooding is an extremely important part of this section. Some sites may have several damage elevations. Provide the elevations and a brief description of the potential damage in this section. Attach a sketch or the XXXXDIMAP001.DGN file may be used provided the map file clearly specifies property that could be damaged from upstream flooding. Finally, provide digital photos with labels as needed.

The D_{50} is defined as the median diameter of the bed material, or the diameter at which 50 percent of the sizes are smaller. Provide a visual estimation of the D_{50} for the channel bed material. Include a picture with rod for scale if possible.

If the stream goes dry at some time during the average year, it should be so indicated in the comment space provided. In addition, provide comments regarding any known fisheries issues.

11.1.1.4 Additional Comments (Unusual Watershed Characteristics)

Sometimes a watershed is encountered whose characteristics differ significantly from those expected for watersheds in that vicinity. When an unusual characteristic is encountered, it will be the responsibility of the survey personnel to write a detailed description of the unusual feature and attach it to Form HYD-1. Following is a brief discussion of several unusual characteristics that might be encountered and what information should be given about them.

It is difficult to make runoff predictions for streams that get a great deal of their flow from springs. When such streams are encountered, the location of the springs within the drainage basin and an estimate of their flow should be given. Because of the difficulty in predicting runoff for such a stream, descriptions of other crossings both upstream and downstream should be given. The following information should be given in the description: size of pipe or opening under the bridge, type of entrance, length of pipe, type of pipe, height of fill, and historical adequacy. Upstream and downstream crossings can be county roads or private approaches up to ½ mile away from the project.

Dams can also affect the runoff and complicate runoff predictions. They should be adequately described so their effect can be considered. Large government dams need not be described because records for these dams are readily available. The small stock dams are of primary concern. The description of the dams should include the name of the owner and builder, the type of material (earth or concrete), height, location in drainage basin, approximate area of the reservoir when full, and a detailed description and dimensions of the spillway and outlet control works. Finally, record the location of any known beaver dams in or near the location of the hydraulic survey.

11.1.2 EXISTING STRUCTURES (HYD-1, SECTION 2)

The purpose of the existing structure information is to aid in establishing the size of the proposed structure. The capacity of the old structure, together with the knowledge of how well it has carried past floods, aids in determining the size of the new structure. An existing structure (bridge/culvert) may also influence the hydraulics at the proposed crossing if it is left in place. See Form 11-1 for a completed HYD-1 Existing Structures example.

11.1.2.1 Culverts

Provide culvert inlet and outlet invert elevations in the spaces provided. The centerline of roadway elevation is the centerline elevation immediately above the culvert; and, the overtopping elevation is the roadway centerline elevation at a nearby sag, the elevation of a ditch block, or the elevation of a watershed divide. Finally, indicate the type (if any) of end treatments, left and right, for the culvert. (i.e. FETS, bevels, edge protection, wingwalls, headwalls, aprons, etc.) Refer to the Culverts section of this chapter for survey requirements on existing culverts not requiring a complete HYD-1 survey.

11.1.2.2 Bridges

Examples of bridge types include: concrete slab, prestressed concrete girder, steel truss-thru, steel truss-deck, timber stringer, etc. Provide the number of spans and lengths of each span in the spaces provided.

Include a drawing, sketch, or digital photo with photo editing to describe the following items: low beam elevations for each span, top of barrier rail elevations at each end, and BRCOR elevations at each end and at each pier. In addition, include pier spacing, pier width, pier shape, and channel shape with elevations.

The XXXXDIMAP001.DGN file may be used as a site sketch provided the map file clearly specifies property that could be damaged from upstream flooding.

11.1.2.3 Erosion Data

Scour is defined as the erosion or removal of streambed or bank material from bridge foundations due to flowing water and includes the following types: long-term bed degradation (channel), contraction (abutment), and local scour (piers).

The channel stability information refers to the channel bottom and not the banks. A degrading channel (head-cutting) is one in which the bottom is being eroded away and is lowering. An aggrading channel is one in which the water is depositing material and is building up.

11.1.3 IRRIGATION DATA (HYD-1, SECTION 3)

Anytime a proposed construction project will cross or in any way affect an irrigation ditch, an irrigation survey should be made. Irrigation HYD-1 locations will be specified in the LHSR and Survey Request Form. Form HYD-1 should be filled out completely with the exception of Section 1, Water Crossing Information. The Water Crossing Information does not apply to irrigation crossings. See Form 11-1 for a completed HYD-1 Irrigation example.

Sections 2 and 3 must be completed accurately for irrigation surveys. If all irrigation information requested was included in the LHSR, this should be noted in the comment field. Much of the information requested in Section 3 will require some research but it is extremely important and must be completely filled in. Because the ditch company must always be contacted for approval of plans, the name of the ditch and the name and address of the owner should be provided. Many times a government agency such as the U.S. Bureau of Reclamation, U.S. Bureau of Indian Affairs, or the Montana Department Resources and Conservation has control and jurisdiction over an irrigation district. If this is the case, the name of the agency concerned should also be provided.

The capacity requested in Section 3, should be that provided by the ditch company and not the surveyed values. If the ditch carries floodwater, this should be indicated and an explanation attached.

The XXXXDIMAP001.DGN file should contain all of the information required for natural drainages plus a layout of the entire irrigation system within the scope of the PFR or LHSR. All main canals, laterals, field ditches, and their direction of flow should be shown. All hydraulic structures such as pipes, flumes, turnouts, drops, checks, and headgates should be located. The invert elevations of these structures should be indicated. Note any washout areas.

The cross sections taken for irrigation should be typical sections that will represent a section of the ditch. Cross sections should be taken when channel characteristics, such as shape, area, vegetation, and channel material change. The location of each section and the reach of the ditch for which each section is typical should be indicated on the XXXXDIMAP001.DGN file. Vegetation and channel bottom material should be accurately described so the channel roughness can be determined. Digital Photographs of each cross section should be indexed in a spreadsheet format.

A profile of the invert and a profile of the water surface should be provided for each ditch. Slopes as flat as 0.0002 feet per feet are not unusual for irrigation ditches so it may be necessary to extend the profiles at least 1500 ft (450m) downstream and measure elevations to hundredths of a foot (thousandths of a meter) in order to establish the slope. The invert elevations of structures, such as turnouts, checks, etc., should be recorded. If a channel change is required, a profile should be run along the approximate channel change line.

The Plan View should be completed in the same manner as for natural watersheds. If the irrigation systems are complex and are to be involved in a revamping, or a possible relocation of the irrigation, the irrigation ditches and facilities should be mapped out far enough so that an adequate design can be made. See Figure 11-1 Irrigation Survey for Plan View example.

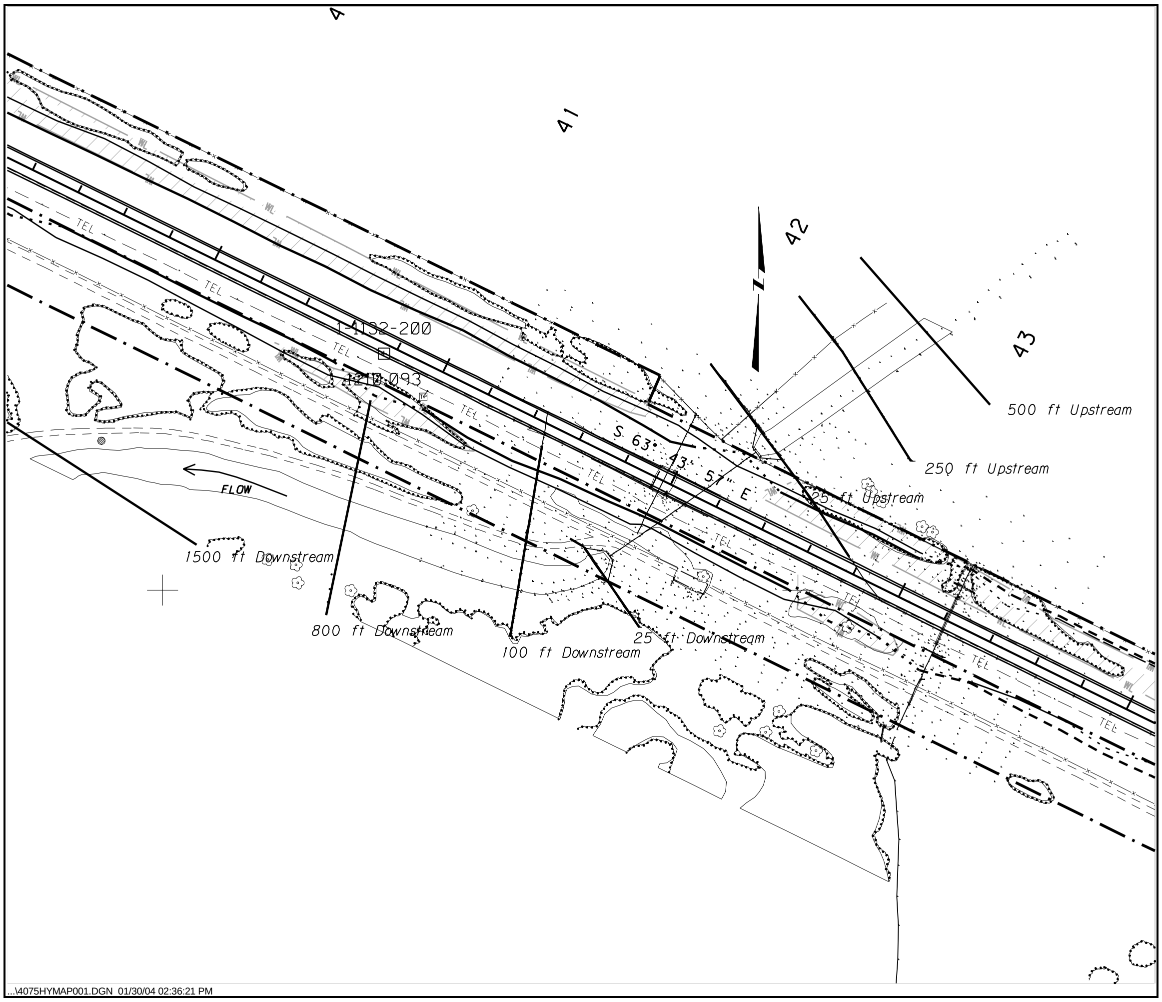

Figure 11-1
Irrigation Survey

11.1.4 WATERWAY CROSS SECTIONS (HYD-1)

The channel cross sections are the most important part of the survey data collected. Care should be taken to see that they accurately describe the channel through a reach of 500 ft (150m) upstream to at least 1500 ft (450m) downstream, or as specified in the LHSR. Include cross section data in the XXXXDIMAP001.DGN file, create the XXXXDINEZ001.XYZ file, and transfer them to DMS.

Cross sections must be referenced from left to right facing downstream. This will be accomplished by adding a comment at the beginning and end of each cross section. (Example: XSECT:LT 500 ft US and XSECT:RT 500 ft US) LT is used to designate the beginning of the cross section and RT is used to designate the end of the cross section facing downstream. The data may be collected in any sequence; however, the data points must be graphically connected from left to right for each cross section and each cross section must have a unique chain name.

Three map files are included at the end of this section for reference. The example map files are as follows:

- Figure 11-2. HYD-1 With Roadway Photogrammetry (Drainage)
- Figure 11-3. HYD-1 With Photogrammetry (Irrigation)
- Figure 11-4. HYD-1 With Data Collector Survey Only (Drainage)

Each cross section should be taken perpendicular to the direction of the flow at each flow stage (see Figure 11-2). It is frequently necessary to bend cross sections to maintain a perpendicular orientation at higher flood events. Since the area of flow is determined from the cross sections, an improperly oriented section may produce a large error. Each section should be extended laterally far enough to extend above the maximum possible flood elevation. Sometimes this will require a cross section of a very wide flood plain, but this is information that is required and must be taken. Contact the District Hydraulic Engineer with any questions regarding flood estimations. If a dtm map is available across the entire floodplain, either from photogrammetric mapping or field survey, cross sections should only extend from the thalweg to 100-ft (30-m) beyond the left and right bank. Photogrammetry aided cross-sections must also be extended in locations of ground obscuring vegetation (void locations). (See Figure 11-3.)

Data points should be collected at all significant changes in slope; and data point spacing must not exceed 25-ft (10-m). Increase the density of points collected for each cross section in and around the bank and channel locations. Cross sections must include enough points to accurately describe the channel at the cross section location. A minimum of five (5) data points shall be collected in the active channel.

Another very important feature of a cross section is the location and classification of vegetation and streambed material. Changes in vegetation and/or streambed material should be indicated in the comment field when data is collected. (Example: XSECT:Hvy Brush to Crop.) Further, comments must be included when the following features are encountered:

- top of bank (XSECT:TOB LT)

- edge of vegetation (XSECT:EDGEVEG LT)
- edge of water (XSECT:EDGEWAT LT)
- bottom of bank (XSECT:BOB LT)
- thalweg (XSECT:TW)

The first and last point of the final graphical cross section shall indicate the distance up/down stream and the chain name for that cross section by placing a notation in the description field.

The LHSR will state the minimum cross sections to be collected; however, the following list should be used as a guideline for the collection of additional cross sections:

- at distinct changes in stream bed slope
- immediately upstream and downstream of pools and riffles
- to accurately describe variations in geometry, including abrupt expansions and contractions in channel or floodplain width
- to accurately describe variations in channel and overbank resistance (vegetation)
- at bends in the stream to ensure that channel and overbank reach lengths are correctly defined
- beneath any existing bridges in place

Digital photographs provide an excellent reference for the Hydraulic Engineer during design. Survey crews must determine the best location for cross section photographs. Each cross section photograph should depict the overbank vegetation and channel characteristics. Additional photographs may be required at each cross section. As stated above, label all photographs and include an index of all digital photos in a spreadsheet format. The photos should be taken at a time when the vegetation and channel are not obscured by snow or ice, if possible. Locations and directions of all photographs will need to be indicated on the plan view.

The survey crew will need to record the location of each photograph during the data collection process. Refer to the survey help guide, *Data Collection System – Feature Code Summary*, for additional information regarding photo locations.

11.1.5 PROFILES (HYD-1)

The profiles are needed to determine the energy gradient of the channel and to aid in establishing the invert elevations of the required structures. The data points for the thalweg, the edge of water surface elevations, and the edge of vegetation on each side of the waterway shall be collected from 500 ft (150m) (or two bridge lengths, whichever is greater) upstream to 1500 ft (500m) downstream (or to the distance called out in the LHSR and/or the PFR) of all structures. This shall be accomplished in a distance interval consistent with the terrain and standard surveying procedures but at no time shall the distance interval for the individual line strings be greater than 25 ft (10m). The thalweg, the water surface elevation, and the edge of water line strings shall include data points directly beneath the centerline of the structure.

The thalweg profile of such rivers as the Missouri, Yellowstone, and Clarks Fork are very difficult and may be omitted if an adequate water surface profile is taken. A detailed survey is still required 130 ft (40m) upstream and downstream of the structure.

11.1.6 PLAN VIEW (HYD-1)

The plan view section serves two purposes: (1) the hydraulic engineer uses it to determine some of the hydraulic features of the crossing, such as the basin overtopping elevation, and (2) the Bridge Bureau uses it to locate new bridge ends when a bridge is required. The XXXXDIMAP001.DGN file is now used as the plan view for hydraulic and bridge design.

Provide sufficient density of data collector survey shots 130 ft (40m) upstream and downstream of the crossing to accurately provide a site map with 1.0 ft (0.50m) contours. This is typically accomplished by collecting data along all break lines and in a grid pattern throughout flat areas and within the active channel. Spacing between data points within the grid should not exceed 25 ft (10m). The grid pattern should extend across the entire floodplain similar to the cross section width described above.

Provide a roadway centerline profile (cross-section) to delineate the potential overtopping location(s). Provide shots at 25 ft (10m) intervals along the existing roadway centerline 400 ft (120m) in each direction including bridge end locations. Extend roadway centerline shots to the vertical sag location if the vertical sag is located within the floodplain. Include a shot at each end of any solid barrier rail if present.

The survey crew shall collect data points for surrounding property as requested in this chapter. The data points collected include, but are not limited to the following: culverts, private approaches, railroad features, structural features, utility features, irrigation canals, irrigation structures, agricultural property, etc.

Provide digital photos with dimensions of any unusual structures such as, but not limited to the following:

- division boxes
- head gates
- weir structures
- turnouts
- drop structures
- reservoirs
- spillways
- downstream pipes or bridges

As stated at the beginning of this chapter, label digital photos using a photo editor program and include a digital photo index in spreadsheet form with each hydraulic survey.

The surveyor shall attempt to coordinate with the Road Designer to review the design file XXXXDIMAP001.DGN representing the collected information. As specified in Sections 1 and 2 of this chapter, property that could be damaged from upstream flooding shall be clearly labeled.

The labels shown in Figure 11-2 are for reference only. Each cross section shall be labeled in the description field as specified in the Waterway Cross Sections portion of this chapter. Photo locations shall be added to the XXXXDIMAP001.DGN file by use of the PHOTO feature code.

The designer shall create contours on 1.0 ft (0.5m) intervals and place them on Level 20 and the design alignment (if available) and annotation shall be placed on Levels 3 and 4, respectively.

The surveyor must insure that all line strings (especially dtm features), for the roadway, terminate at the bridge ends (guard angle) and not at the preceding data point. They must also collect the data points directly under the structure to indicate the ground relief around the end bents and wing walls. If the data points are not properly collected, the designer cannot accurately create the necessary contours at the bridge ends. This information is vital to calculating the existing structure's opening area.

The LHSR or the PFR will provide site-specific information and requirements. Contact the District Hydraulic Engineer with requests for information not provided in this chapter, or the LHSR.

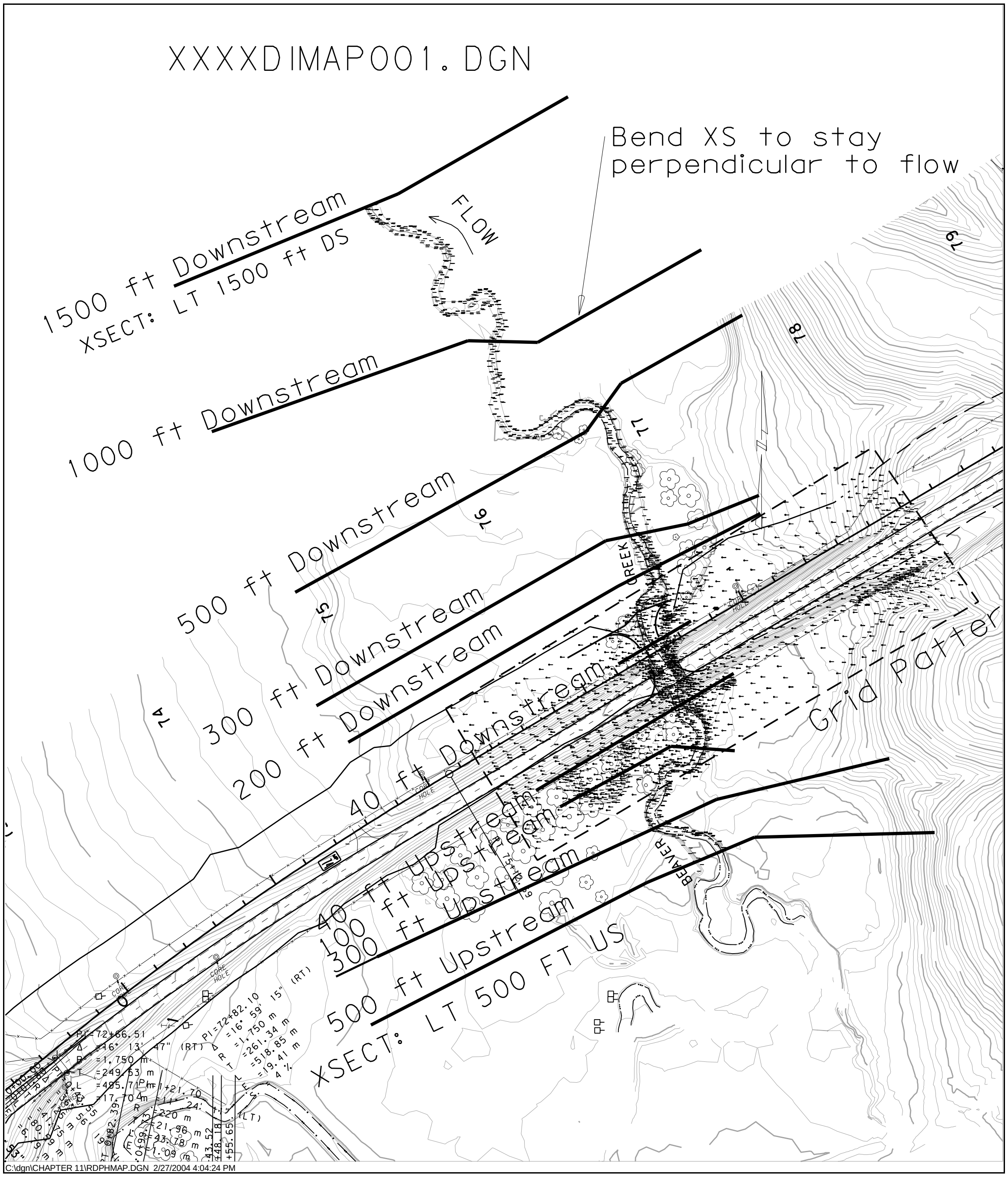

Figure 11-2
HYD-1 With Roadway Photogrammetry (Drainage)

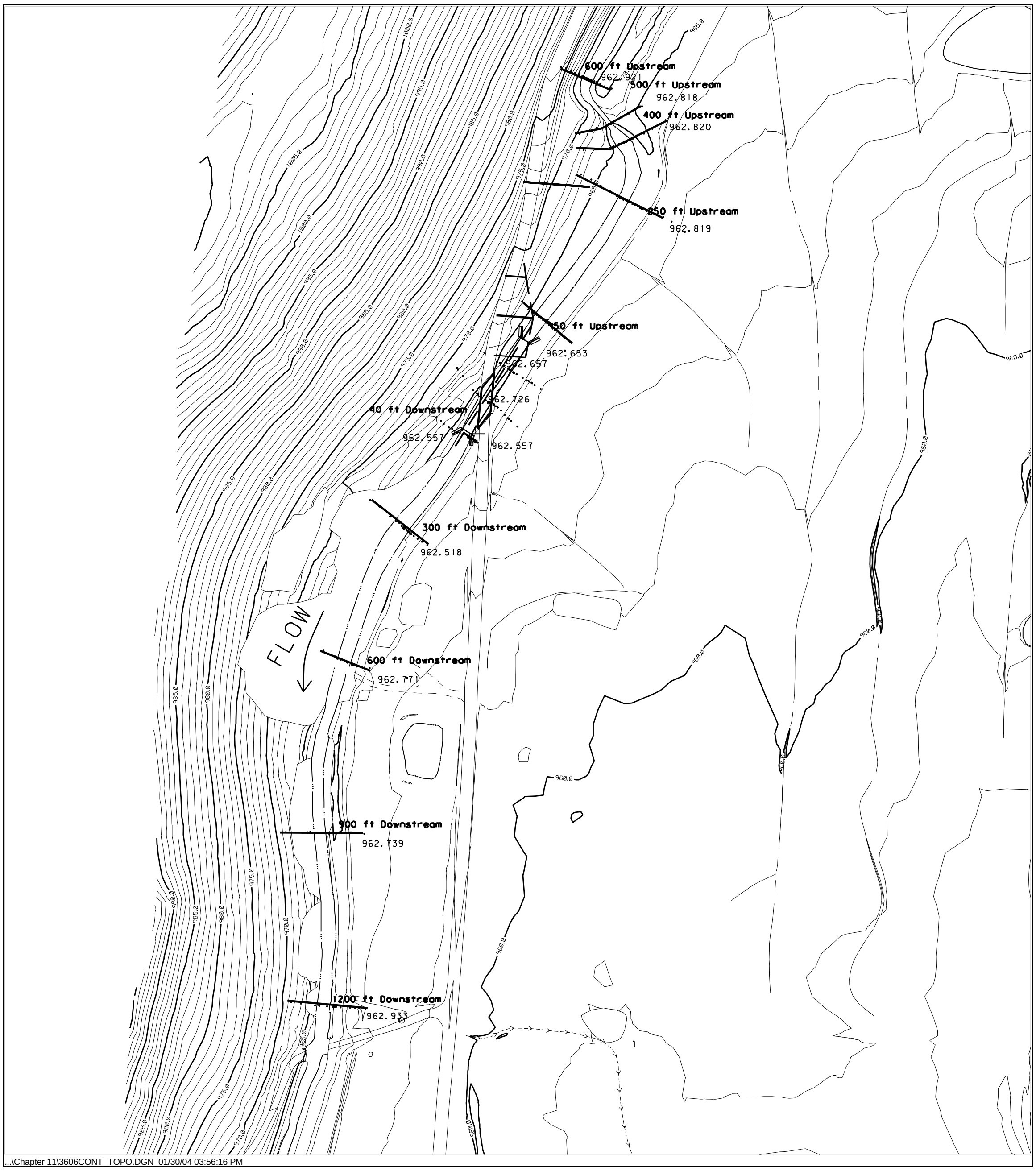

Figure 11-3
HYD-1 With Photogrammetry (Irrigation)

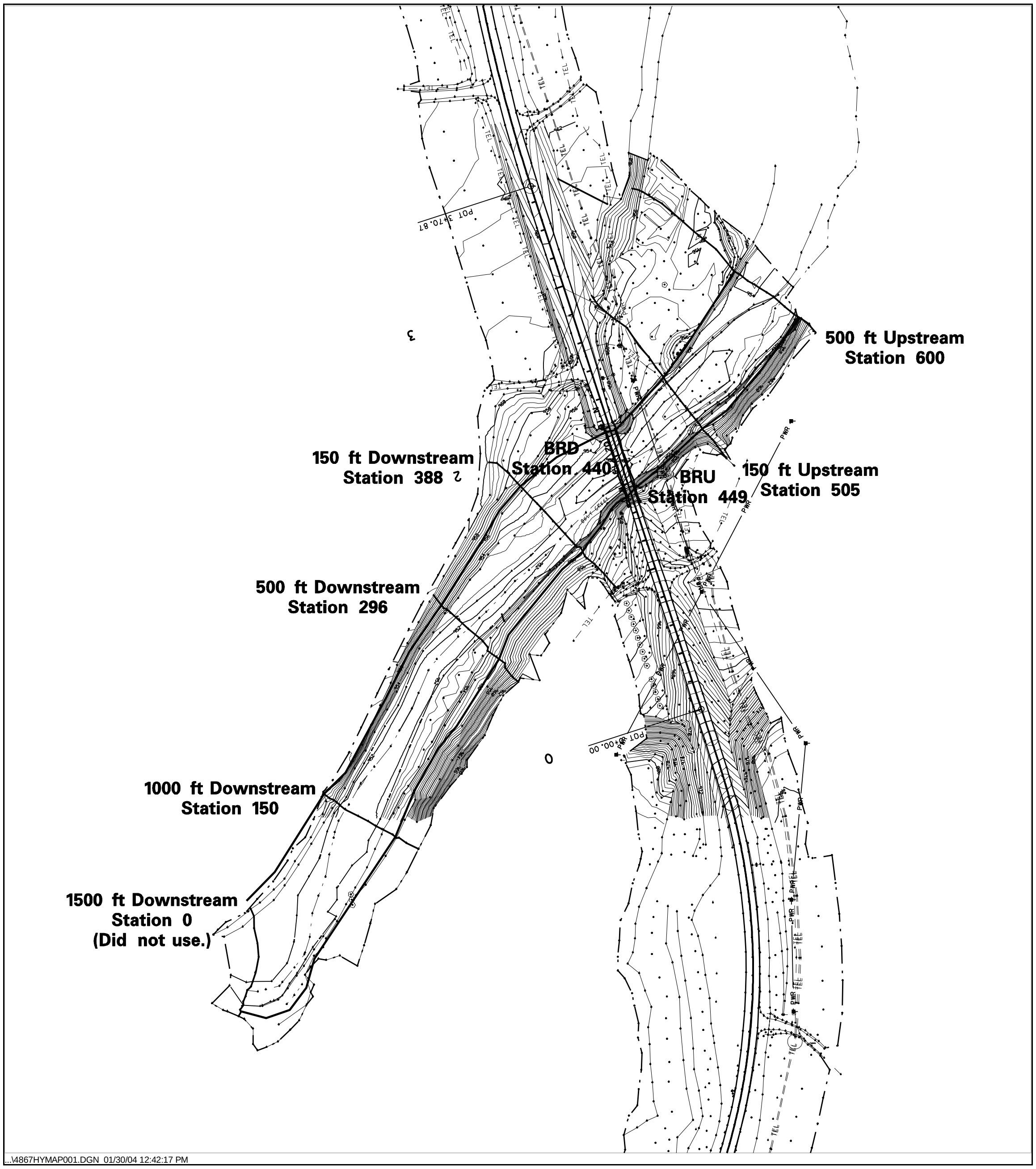

Figure 11-4
HYD-1 With Data Collector Survey Only (Drainage)

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11.2 URBAN STORM DRAINAGE

The information required to design a storm drain is extensive and variable. This makes it difficult to give a complete and accurate description of the needed information. The intent here is to present a general idea of what is needed and why it is needed. Contact the District Hydraulic Engineer with any additional questions regarding storm drain surveys.

11.2.1 CONTRIBUTING AREA

Determining the area that contributes to the storm drain runoff is the most difficult part of the storm drain survey. Most storm drains pick up water that runs onto the highway from side streets. Data should be collected to produce a profile of the gutter flow line up side streets for a minimum of 200 feet (70m) or ½ block.

A detailed description of side drainages should be given so drainage areas and runoff coefficients can be determined. The description should include such information as drainage limits, direction of flow on each street, street widths, type of surfacing, land use (residential, commercial, etc.), location at which it intercepts the highway, and location and size of any large paved areas such as parking lots. If there is an existing storm drain in place in any of the contributing areas, the location, size, and elevations of all inlets, manholes, and pipes should be given.

Most needed information can easily be included in the XXXXDIMAP001.DGN file with the help of Road Design. Any other information that could be pertinent to the design of the storm drain system should be included.

11.2.2 OUTFALLS

It is always necessary to discharge the storm drain into a natural drainage at some point or points along the system. The survey crew should work with the Hydraulic Section to identify possible outfall lines and provide a survey of each line. Some storm drains do not have an obvious outfall line, and a very detailed search is required. The outfall survey should include all pertinent topography and a channel profile. Possible outfall options include: grass lined ditches; natural swales; wetlands; and settling basins.

11.2.3 UTILITIES

All buried utilities that could affect the installation of the storm drain must be accurately located. The size, depth, and location of each utility must be recorded. The depth given should be referenced to the project datum. The invert elevations of all manholes and connections on gravity systems such as sanitary sewers should be given if possible. Contact the utility companies and city to get most of this information. If for some reason they cannot provide the needed information, some fieldwork may be necessary. If Sub-surface Utility Engineering (SUE) survey is requested in the LHSR, sub-surface data collector survey is not required for utilities.

11.3 CULVERT SURVEYS

All drainage crossings regardless of culvert size must be surveyed. This section describes the requirements for the data collection of culverts (non-irrigation) not designated for a HYD-1 survey. Refer to the LHSR or the PFR for a listing of drainage crossings requiring a HYD-1 survey. As stated above, contact the District Hydraulics Engineer with questions regarding drainage crossings not discussed in this chapter, the LHSR, or the PFR.

The following list includes current feature codes and attributes. Please refer to the latest revision of the feature code list for updates. Each culvert survey will include some or all of the following data points:

- CULVI (culvert invert shot at end of pipe and at end of culvert terminal section)
    - Type (specify material and primary function of culvert)
    - Culvert End (specify type of terminal section)
    - Cutoff Wall (yes or no)
    - Edge Protection (concrete, riprap, other, or none)
    - Damaged End (yes or no)
    - Clean (specify percentage filled with sediment)
- CULVT (shot on top of culvert at each end of pipe)
- FL (collect flow line data points in the direction of flow)
- FLU (collect data points against direction of flow)
- XSECT (collect XS data points through channel/swale at proposed R/W line)
- DTCBLK (collect data point at low point on ditch block)
- PTW (shot at centerline of roadway above culvert)

Frequently, culvert inverts are buried under snow, ice, sediment, and debris. These obstructions must be removed if possible prior to collecting data. The true culvert inlet invert and outlet invert (CULVI) are located at each end of the culvert (not at the end of the terminal section). Data shots collected at the end of the terminal section (i.e. FETS) should be noted in the comment field of the CULVI feature code.

The flow line should follow the natural channel/swale from the upstream side of the estimated construction limits, through the culvert (if possible), and terminate at the downstream estimated construction limits. Data points should be collected at all changes in flow line grade; and spacing for flow line data points should never exceed 25 ft (10m). A flow line data point should be collected at 1 ft (0.3m) beyond the upstream and downstream end of each culvert/FETS. Indicate dimensions of scour holes in comment field if located. Use MISCP to take shots around scour holes. Do not use the FLU feature code unless collecting the flow line against the direction of flow.

Provide one cross section perpendicular to the flow (best estimate) at the estimated construction limits, right and left, for culverts larger than 3.0 ft (900mm) in diameter. Cross sections should extend beyond known high water marks for each crossing.

A ditch block is usually a small earthen mound placed across the roadway ditch immediately downstream of a culvert. Ditch blocks are frequently used to create more headwater at a culvert inlet. Collect the DTCBLK data point on top of the ditch block, at the sag location. Finally, use the PTW to provide a shot on centerline of the PTW above the culvert to provide a roadway overtop elevation.

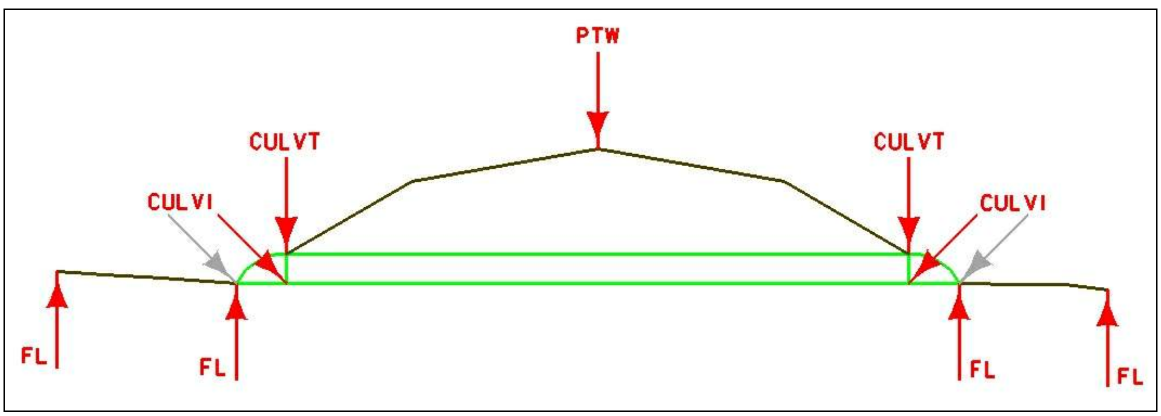

Figure 11-5
Culvert Survey
(Collect Flow Line to Proposed Construction Limits)

11.4 SMALL EARTH FILL DAMS

It is sometimes possible to use a highway fill as a small dam and back up water either for flood routing purposes or for use as a stock pond. This situation usually requires a crossing with a small drainage area (ten square miles or less) and a high fill. In order to safely design such a crossing, some special information is required in addition to a completed Form HYD-1. Refer to the LHSR for more detailed information regarding survey requirements.

It is necessary to determine the storage capacity of the reservoir that will build up behind the fill. In order to do this, a contour map of the area upstream is required. A contour interval of five feet is normally sufficient. The contours should go as high as the top of the fill.

The location and elevation of anything on the upstream and downstream side of the fill that could be flooded and damaged should be recorded. Some description of the land and vegetation that could be flooded should also be provided.

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11.5 ROADSIDE EROSION

Erosion in roadside ditches, median ditches, on cut and fill slopes, etc., is a problem that should be handled during the original design. The best way to recognize potential erosion situations is by reviewing similar situations in the immediate vicinity.

Take note of any erosion in existing roadside ditches, on cut and fill slopes, on hillsides, and in nearby gullies. If these locations show signs of erosion, the potential for erosion of the newly exposed slopes of the new highway is great. The survey crew should identify these locations by adding comments to the readme file.

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Form HYD-1

MONTANA DEPARTMENT OF TRANSPORTATION

HIGHWAYS DIVISION

Survey Data for Design of Waterways

Project: Lohman - East & West — No: F 1-7(11)394

Waterway Name: Milk River — Station: R.P. 397.772 (FAGH 66 205+59.50)

County: Blaine — Date of Survey: 11-24-00

Surveyed By: Olson & Zinn — Culvert Size: 1.219m

THIS FORM SHALL BE COMPLETED FOR ALL WATERWAYS AND CROSSINGS AS DESIGNATED BY THE LHSR

GENERAL INSTRUCTIONS

1. Fill out all blanks carefully, giving all information obtainable. (See Chapter 11 (Hydraulic Surveys) of the MDT Surveying Manual for more detailed instructions, discussions, and examples.)

2. Make any comments or provide any additional information that you feel may be pertinent to the design of this waterway crossing. (Submit all pertinent information with this form including but not limited to sketches, digital photos, reconnaissance, etc.)

3. Comments of local residents will aid in filling out much of this information.

4. The ditch rider, farmer, or ditch company should be contacted when filling out Section 3. Attach any R/W agreements as applicable.

5. Use additional forms and sheets as necessary.

6. Submit digital photos of all channels and overbanking areas, and all existing structures. Be sure they are well labeled. Indicate the location and direction of all photographs with an arrow (→) on the plan view. The minimum required photos are listed as follows:
   a) Up & down line of the roadway from each structure.
   b) Bridge opening downstream of each structure from each side of waterway.
   c) Bridge opening upstream of each structure from each side of waterway.
   d) Downstream from on top of each structure.
   e) Upstream from on top of each structure.
   f) Waterway at each cross-section from each side of waterway.
   g) Inlet and Outlet of each drainage system.
   h) All irrigation system structures.
   i) Material type (D50).
   j) Any difficult to describe areas of concern.

Form 11-1
Metric Example-Use Project Units

Section 1 -- WATER CROSSING INFORMATION

Floating Debris: ☐ None ☒ Light ☐ Medium ☐ Heavy ☐ Other (_____)

Size of Debris: Diameter: 75mm Length: 2.0m
Describe: Limbs and sticks. Brush hung up on pier.
(log, brush, other – provide photos)

Ice conditions: ☐ Light ☒ Moderate ☐ Severe
Thickness (mm) (0 – 150) (150 – 300) (300 – 600)

Comments: Mr. John Smith (Adjacent Landowner) indicates he has seen ice 1-foot thick at crossing.

Water Surface Elevations at Crossing

Elevation of Water on Date of Survey: 736.91m

"Normal" water surface elevation (Vegetation/Bench Line): 739.77m ☐ Normally Dry

Elevation of "Historical" High Water: 741.63m Date Occurred: 3-17-75

Cause: ☒ Backwater from Ice ☐ Backwater from Construction ☐ Cloudburst ☐ Spring Runoff

Structure overtopping: ☒ No ☐ Yes
If yes, Frequency N/A

Comments: See digital photo #1010 for high water mark on pier.

Will ponding of water at crossing cause damage to buildings or other property? ☐ No ☒ Yes
(Include sketch, map file with labels, & digital photos as necessary.)

If so, what elevation will water be allowed to reach (floor or doorway elevation): 742.00m

Comments: See note on DIMAP for flood risk location.

D50 Estimation

Material:	☒ Silt/Clay	☐ Sand	☐ Gravel	☐ Cobbles	☐ Boulders
Size range (mm):	(<0.062)	(0.062 - 2.00)	(2.00 – 64)	(64 – 250)	(>250)

(Include digital photo with rod for scale if possible.)

Comments: See digital photo #1012 for D50

Form 11-1
Metric Example-Use Project Units (cont.)

Section 2 -- EXISTING STRUCTURE

1. Culvert Data

Inlet Invert Elevation: 732.215m
Outlet Invert Elevation: 735.201m
Centerline Rdwy: 737.854m
End Treatment LT: 2:1 Step Bevel
Overtopping Elevation: 736.850m (roadway/divide)
End Treatment RT: 2:1 Step Bevel

Comments: Overtopping occurs at ditch block to east. A cutoff wall and concrete edge protection are located on the inlet and outlet ends of the culvert. Random riprap is located at the end of the culvert. The culvert is 1/2 full of sediment. See point numbers 13001 through 13006 for additional culvert data.

2. Bridge Data

Bridge Type: Steel Pony Truss

No of Spans: 2

Length of each span: 1 *See Note 2 ____ 3 ____ 4 ____
5 ____ 6 ____ 7 ____ 8 ____

Comments:*See attached bridge sketch for span lengths and point numbers.

Include a drawing, sketch, or digital photo with labels to describe the following (see attached diagram):

- Low beam elevation at each bridge end and at all piers
- Pier spacing
- Pier width
- Channel shape w/ elevations

Include a site map sketch (map file with labels) showing locations of:

- Sections
- Roadways
- Buildings
- Unusual structures or items

3. Erosion data

Do banks or bed show scour? ☐ No ☒ Yes

Evidence of pier scour? ☐ No ☒ Yes

Is the channel: Stable ☐ Degrading ☒ Aggrading ☒ Unknown ☐

Comments: Channel is degrading upstream and aggrading downstream of crossing (sand bar).

Form 11-1
Metric Example-Use Project Units (cont.)

Section 3 -- IRRIGATION DATA

Name of Ditch: USBR Lateral 27.8 Owner: Helena Valley Irrigation District

Address of Ditch Company: HVID, John Doe, Manager, 3840 N Montana Ave, Helena, MT 59602

Ditch Capacity
Owner's opinion: 1.00m^3/s Irrigated Acres: 500 Acres

Provide sketches or digital photos of any additional structures such as:

1. Head Gates
2. Screw or Slide Gates
3. Turnouts
4. Check or Diversion Structures
5. Division Boxes
6. Drop Structures

Comments: The irrigation season is from April 15 to September 30. The ditch rider's name is Jack Doe. See photo #2000 thru 2020 for irrigation structures along Lateral 27.8. See DIMAP for cross section and Photo locations.

Additional Comments & Notes:

Form 11-1
Metric Example-Use Project Units (cont.)

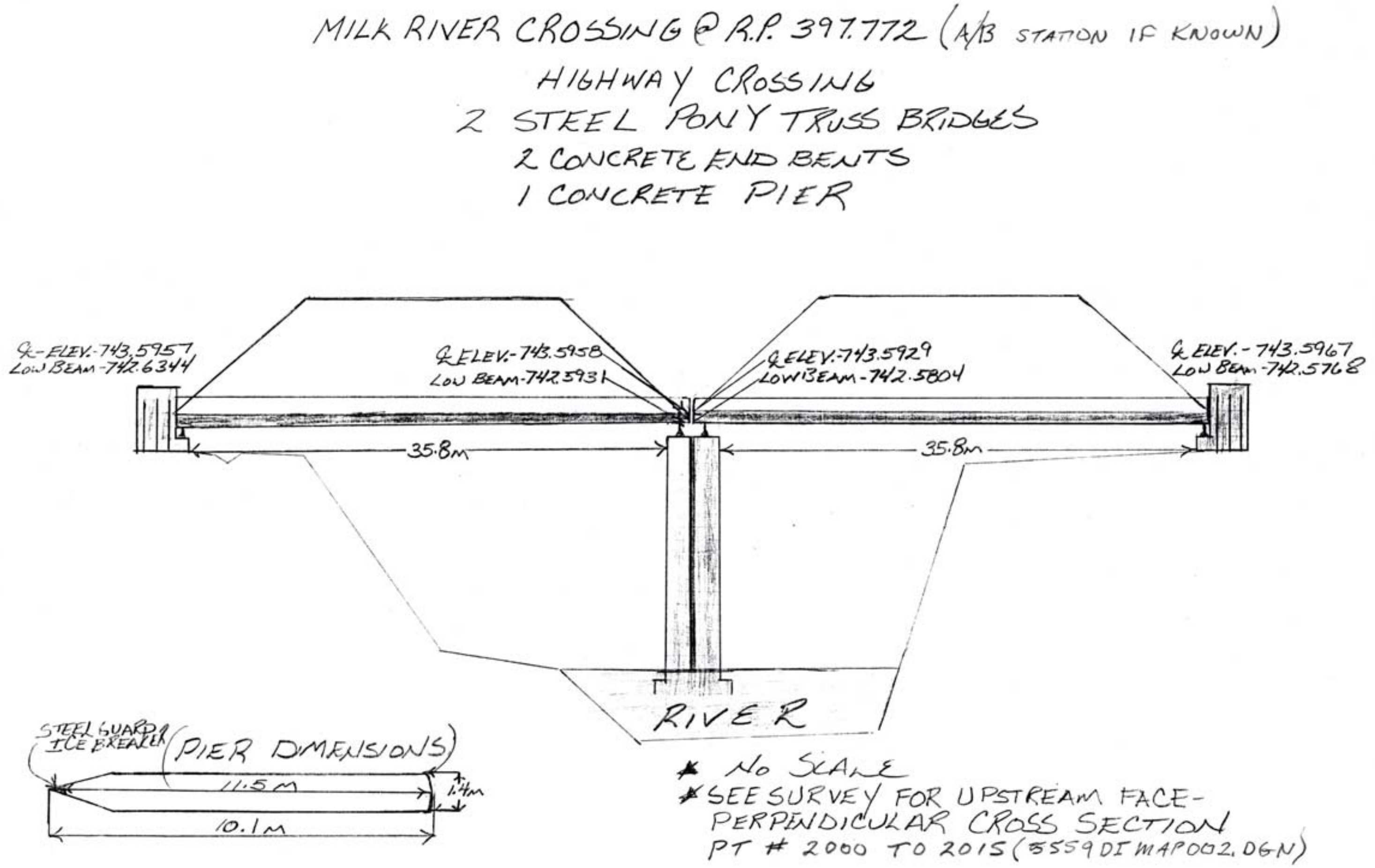

Figure 11-6
SAMPLE BRIDGE SKETCH
(Metric Example – Use Project Units)

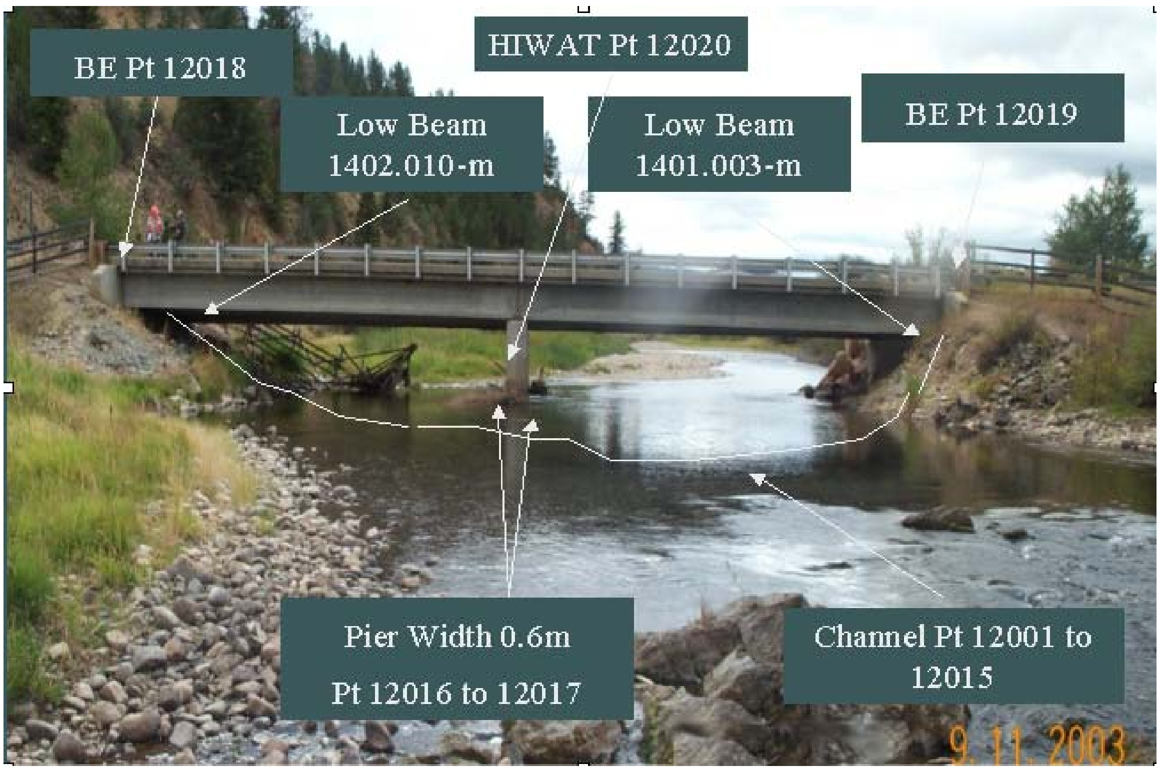

Figure 11-7
Bridge Sketch (Digital Photo Option)
(Metric Example – Use Project Units)

Chapter 12
Field Notes

Table of Contents

Section **Page**

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Chapter 12
Field Notes

12.1 SURVEY NOTES

Survey field notes are recorded at the time the observation is made. Field notes may be recorded in a standard field book (conventional), recorded in a data collector file, or a combination of each. The field notes should represent a complete and accurate record of the survey. Conventional survey notes should include:

- index
- names and duties of each survey crew member
- instrument model(s) and serial number(s)
- survey location
- diagrams or sketches
- pertinent record information and references
- consecutive page numbers in the upper right hand corner
- original “raw” data values (without any mathematical manipulations and without any corrections for errors) of distance, angle, and elevations
- recording of pertinent atmospheric conditions, such as temperature, pressure and ppm settings
- monuments found or set, and complete descriptions e.g. found 5/8-inch rebar with 1 1/2-inch plastic yellow cap, stamped 4378S, SE corner per COS 1234.
- title page for each day
- explanatory notes about any condition(s) that might affect the accuracy or result of the survey and
- any items the survey crew feels are pertinent

Field notes for Department projects, in addition to the above will include on the title page, the control number (CN) also known as the uniform project number (UPN), the project identification, and location of the project.

Field notes associated with cadastral surveys, in addition to the above, should include:

- statements by adjoining landowners relative to boundary line and corner locations
- corner search and monumentation documentation

The widespread use of data collectors and total stations has replaced some of the major purposes of conventional field notes. The information contained in data collector files is generally limited to the recording of raw observations and comments. Conventional field notes should include the file name of any corresponding data collector files.

12.1.1 IMPORTANCE

Field notes are an integral part of any survey. A survey is never completed until field notes and data collector files are submitted for checking and filing. Field notes perpetuate a survey even when stakes have rotted and monuments are obliterated.

Conventional field notes, together with data collector files, diaries and survey party reports, are occasionally the primary source of the Department's documentation in court cases arising between the Department and landowners or contractors. For this reason, measured values should ***never*** to be erased. If a numerical value is wrong, cross it out. Original field notes should not be copied, unless there is a very good reason. Copied notes will always be labeled “COPY,” and the reason will be shown. If notes are copied, the originals will always be retained.

In view of the importance of the field notes, the duties should always be assigned to a knowledgeable member of the party. The note keeper should have a thorough understanding of the purpose of the survey and the operations.

Survey field notes serve as an interpretable field record of a survey party's every step in the prosecution of a survey.

12.1.1.1 Uses

Conventional field notes in conjunction with data collector files are used by the survey party to record all measurements made in the field and to show lines and points established or found.

Survey notes are often used by others to check the accuracy of the survey, adjust the survey to derive the best values, and to extract data for other surveys. They are also the primary basis for most design, traffic, right-of-way, construction, and other engineering measurements.

12.2 NOTE KEEPING

12.2.1 ELEMENTS OF NOTES

12.2.1.1 Title Page

Filing and subsequent use of field notes is facilitated by a title page. A title page contains a summary for a set of field notes. Each day's notes will begin with a title page. Information included on the title page should identify the project number, control number, type of survey, crew members and their duties, instruments used including serial numbers, and weather conditions. Other information that will aid someone searching for specific survey information may also be added. By the time a project survey is completed, there may be numerous books and forms filed. Well-indexed notes will save time and expense.

12.2.1.2 Sketches, Notes and Diagrams

Some entries are made primarily for aiding in the interpretation of the measured values. These include:

- a north arrow, always shown on each page where a sketch is used
- planimetric features
- standard mapping delineation lines, symbols, numerals, and letters and
- descriptive notes and datums

12.2.2 RECORD INFORMATION

Record data is defined as that information that can be retrieved from formal filing systems. Some samples of record data are: coordinates, stations, survey data, point descriptions, bearings, or azimuths (with basis given), and elevations. Record Department information can include control survey diagrams, field survey notes, right-of-way and construction plans. Other sources of record information may include original General Land Office (GLO) field notes and plats from the Bureau of Land Management (BLM), datasheets from the National Geodetic Survey (NGS), and corner recordation's, deeds, certificates of survey from county records.

Most of the record information should be assembled prior to the field survey. Where practical, the source should be cited for the acceptance and use of any found monument.

A “record monument” is one that is identified as being documented in the public records and has been proven to occupy its original position. The word “found” is always used in describing recovered monuments. However, all found monuments are not necessarily record monuments. The survey crew may find monuments for which they do not have any substantiating information. When survey monuments are set, the field notes should indicate the raw measured values. The notes will describe in detail all record, found and set monuments. Complete descriptions need to be included only once. Any subsequent reference to that monument can refer to the book and page containing the initial detailed description.

Calculated data results from mathematical manipulation of measured values. Whenever a calculated value is shown, the record or measured values that are the basis of the calculation should also be shown. For example, if an angle is measured several times the recording of only the mean is not acceptable. Enter the circle reading for all positions. If an inverse in the field has been made the coordinates of both points should be shown in the field notes.

12.2.3 OBSERVATIONS AND MEASUREMENTS

Besides the actual observations and measurements, field notes should include descriptive information, and point names. The observations and measurement represent the heart of the survey. Record each required field value in its proper place. In addition explanatory notes, such as unusual weather conditions and problems with equipment, aid in the interpretation and analysis of reliability of portions of a survey.

Those entries, which are recorded at the time observations are made, are “original” entries. Record original entries of observations. Field notes with original entries have more authority and will be viewed by the courts as such. After-the-fact entries are suspect and more likely to be erroneous.

Notes should not be recopied. However, in those rare cases when notes are copied for clarity, the original notes must be retained. The originals should be attached to the copied notes.

Problems can arise in recording certain types of observations or in interpreting what has been recorded for any type of observation. Some guidelines to follow are:

- Record measurements that are equivalent to the precision of the equipment and/or surveying techniques.
- Always record the full raw value.

- Do not record only the sums or the differences, or the means of a series of raw values. Also, do not record only “corrected” sums, differences, or means.
- Sea level and grid distances are calculated values. When setting points, show calculations that convert a grid value to a ground value.
- Record significant figures only.
- Clearly indicate the decimal portion of a measured value.
- When different distances or the same distances are measured by more than one method, note the equipment or method used for each measurement.

Do not erase any observation. If a mistake is made in recording, or if an observation is rejected, draw a line through the entry, without destroying the legibility. Erasures diminish, and often destroy, the credibility of field notes.

12.2.4 DESCRIPTIONS

As a minimum the description should indicate the size, type, stampings and markings on the cap, and if the point was found or set. This information will assist in the recovery and the differentiation of that point from any other point. The description may also include drawing of the monument and its location relative to physical features.

12.2.4.1 Importance

The recoverability and positive identification are often in direct relation to the effort expended in originally describing the point. The record points that control a survey must be verified in the notes by descriptions that do their utmost to substantiate the acceptability of the points. Otherwise, the survey could be based on erroneous monuments.

12.2.4.2 Practice

Do not depend on second hand information or verbal description. Write the description while at the point. When record monuments are not recovered, describe how intense the search was and the area covered in the search. Record any physical evidence found of “obliterated” or “lost” monuments or related changes in topography, which might have destroyed a monument and any evidence to it. Describe any physical evidence that might indicate the location of the monument that could not be found.

12.2.4.3 Elements

There are several basic elements that, when included in the description, will aid in its identification in the record:

- designated name of the point such as station, control number, USGS number or name, section, township, range, etc.
- origin of monument (found or set)
- size and materials (stone 14 inches x 24 inches, 2-inch ID pipe, etc.)
- position relative to surface (12 inches under county road surface, etc.)
- condition
- specific marks (stamping, scribing, etc.)
- general location and calls (hillside, west of Rte. 120, 20 feet from S. river bank, etc.)
- document or field notes that first described point

12.2.5 SKETCHES

Sketches are an aid to the interpretation and understanding of the notes by others. Some basic but extremely important rules to follow when including sketches in field notes are:

- Use standard forms when available.
- All field notes should be recorded on standard field notebook paper that is available by requisition from the district office.
- Do not try to economize on paper.
- Work at a speed that is within your ability. If data is relayed to you faster than you can record slow down the person providing the information.
- Do not crowd information to a point where numbers or letters are hard to distinguish or some information is covered.
- Make notes dark enough to be reproducible. Notes should be written with a 3H or 4H pencil or equal. When the lead is too hard the notes are difficult to read and do not microfilm well. When the lead is too soft, the marks tend to smear.
- Use standard abbreviations and symbols. See Appendix C for standard abbreviations.
- Use drafting aids.
- Be consistent in style and lettering.
- Field notes are made by many individuals who use different methods of forming numbers and figures. The essential requirement is that notes be legible and

descriptive. Notes should be recorded so that other surveyors may readily interpret any part.

12.2.5.1 Field Note Drawing

When field notes include detailed drawings, follow these additional suggestions:

- Draw sketches in correct relationship.
- Orient sketches and entries according to standard mapping procedure — that is, have everything read from the bottom or right edge of the sheet.
- Draw the framework of the sketch before measurements begin.
- Do not waste space or time drawing oversize or elaborate north arrows or dimension arrows.

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12.3 CONSTRUCTION SURVEYS

The staking books document the location, dimensions, and materials incorporated in the facility. The notes should include all information required for that documentation. Neat sketches are often a great aid for those who have to read the notes and compute pay item quantities. Although the sketches may not be drawn to the exact scale, they should show actual field measurements. Survey crews should refer to all contract documents and the *Field Office Manual*. The *Field Office Manual* is available from the Construction Bureau.

The construction survey notes cover alignment, bench marks, cross sections, grade stakes, slope stakes, stakes for structures and miscellaneous construction survey data. Survey entries should be the original entry. They are ***not*** to be taken on scratch paper for copying at a later date. All notes shall show the name of the person making the notes, the person doing the checking, the date for each day, the party personnel, and weather information. Symbols may be used to designate the person running the instrument, chaining, rodding or other survey duties.

Each notebook should be identified on the front cover by subject (alignment, cross sections, etc.), project number, control number and the name of the Engineering Project Manager (EPM). On small projects, some separate operations and miscellaneous survey notes may be combined in a single notebook, whereas separate notebooks for each operation may be required on larger projects. Notebooks that contain many different types of survey data should contain an index in the front of the book.

Field books should contain complete information of the constructed centerline or other lines run, reference control points, any errors found in angles or distances, and the corrections made.

12.3.1 HORIZONTAL CONTROL - CENTERLINE

In cases where the EPM requires hand written notes, the alignment notes should show all main control points for the project, all other control points. Prior to entering the information in the alignment notebook, computer output information should be checked against the design plans and any discrepancies resolved.

Normally, line sketches of the alignment, offset line and relative position of control points would be shown on the right-hand page. The stationing should start at the bottom of the page and proceed upward. Each occupied station should be shown, as well as the measured distance and azimuth between such points. It is less congested if only one curve and tangents to and from that curve are shown per page. It is generally more convenient to include curve data on the right-hand page, leaving the left-hand page clear for actual staking notes. Staking notes for the alignment shown on the sketch should be carried on the left page. Such notes would reflect either the traverse stations and deflections, or azimuth and distance to staked station points from control points.

12.3.2 CROSS-SECTION NOTES

All recommended practices are aimed at electronic computation of earthwork quantities while maintaining the simplest records possible. The systems and samples conveyed by this manual have been prepared especially for adaptation to electronic computer operations. However, if all “00s” and intersections of template and ground line are caught and recorded, areas can still be computed directly from the notes. Therefore, it is essential that all construction personnel follow the prescribed patterns as clearly and uniformly as possible. Cross-section notes and data are to be entered and recorded on Form CSN-01. Every set of cross-section data that is compiled by the survey crew should be thoroughly and completely checked by the EPM.

Borrow pit cross-section notes are similar to other survey notes in that the data, weather information, and members of the crew are to be recorded for each day. These notes should be neat and well kept. Figures that are in error should be crossed out and the correct figure written above. Do not erase.

12.3.3 MAJOR STRUCTURES - BRIDGES

Typically, contractors perform bridge surveys. In cases where the Department performs the survey, separate field books should be set up for each major structure and maintained on a daily basis when work is being done on the structure. Information for setting up staking diagrams and sketches should be obtained from the detail sheets in the plans. Separate pages should be used to show the overall staking system and detail drawings of the various structural components. Do not try to crowd too much information on one page.

12.3.4 COMMON ERRORS IN NOTE KEEPING

Construction note keeping errors that occur fairly frequently are discussed in this section.

12.3.4.1 Plus and Minus Rods

A minus rod reading represents an elevation below the height of instrument (HI). A plus rod denotes ground or templates higher that the HI. Plus rod readings are preceded with a plus sign (+).

Normally, it is assumed that if there is no sign in front of a rod reading, it is a minus rod. One exception is illustrated by the following example, where there were a series of plus readings with one or two in between without signs.

$$\frac{+5.6}{19} \quad \frac{+4.1}{25} \quad \frac{+3.6}{28} \quad \frac{+2.8}{30} \quad \frac{1.2}{33} \quad \frac{1.4}{35} \quad \frac{+2.3}{38} \quad \frac{+3.2}{41}$$

It is not clear whether the readings at 33 and 35 are actually minus readings or if the note keeper forgot the + sign. In such a case if the readings are actually minus readings, it is suggested that they be written as: $\frac{-1.2}{33} \quad \frac{-1.4}{35}$

12.3.4.2 Distance Out of Sequence

Occasionally a distance reading is out of proper sequence, such as 00, 10, 15, 25, 20, 31, 36. Are the readings at 25 and 20 reversed? Should the 20 be 28 or 30? Is there a cave or overhang? If the readings are correct, explain. Be sure to edit the notes for such occurrences.

12.3.4.3 Excavation Without Centerline

Occasionally, a borrow area is cross-sectioned without a separate centerline or baseline. This usually occurs as an extension of roadway sections off the roadway centerline. It is essential that every set of cross sections have a centerline or baseline.

12.3.4.4 Wrong HI

Sometimes the notekeeper may, in transferring the HI to the next page, carry forward an incorrect number, such as 3152.69 being carried forward as 4152.69. The error may be carried past a bench mark without being corrected. When data is entered using an erroneous HI, the data will be incorrect.

12.3.4.5 Decimal Points

Now and then a recorder will forget to enter a decimal point in a rod reading or distance, or will enter a decimal point where there should be none.

12.3.5 CORRECTIONS TO NOTES

Engineering Application Programs compute the cross-section data, both template and ground line. On occasion, cross-section notes must be amended, supplemented or otherwise revised after data processing. Remeasure areas and cross-section extensions are two examples.

Changes to records should be made with red pencil so they are easy to find. If the changes are recorded on completely new sheets, such as for borrow widening, the notes need not be in red, but should be clearly marked and noted that the change is to be considered when computing remeasure quantities.

12.4 DISPOSITION AND USE OF NOTES

During the course of a survey, a great deal of existing data sources and newly created survey information is in the care of the survey party. Loss of, or damage to, any of the information can be expensive and cause a considerable loss of time.

12.4.1 HANDLING

Each EPM should make provisions for proper handling and storage of blank forms, notes in progress, and notes that are newly completed. Originals of record notes or those of which there is only one copy should not be sent to the field. Use benchmark summaries, primary control maps, coordinate control maps and listings, and copies of the originals.

12.4.2 INDEXING AND CHECKING

Field notes with raw values and original entries should be indexed before any review is made in the field office. The immediate indexing aids in keeping track of the notes during the field office review prior to submission to headquarters, when applicable.

Field office checks, adjustments and reductions should be made before notes are accepted. Examples of checks are mathematical errors, recordation blunders, traverse closures, spirit level and vertical elevation closures, complete point descriptions, and the taking and recordation of proper data needed for the point described. Examples of adjustments would include spirit level and vertical angle elevation runs. The final adjustment of the traverse needs to be done at this time.

12.4.3 SUBMITTAL

After the completion of a project field notes should be submitted to Helena for archiving. The District should photocopy all original field notes prior to mailing the originals. When original field notes must be mailed, they should be sent by registered mail, with a return receipt requested. Each field book should contain notes for only one type of survey. For example, the notes for bench levels and cross sections should be in two separate books.

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12.5 SAMPLE NOTES

Sample notes are included in Appendix A.

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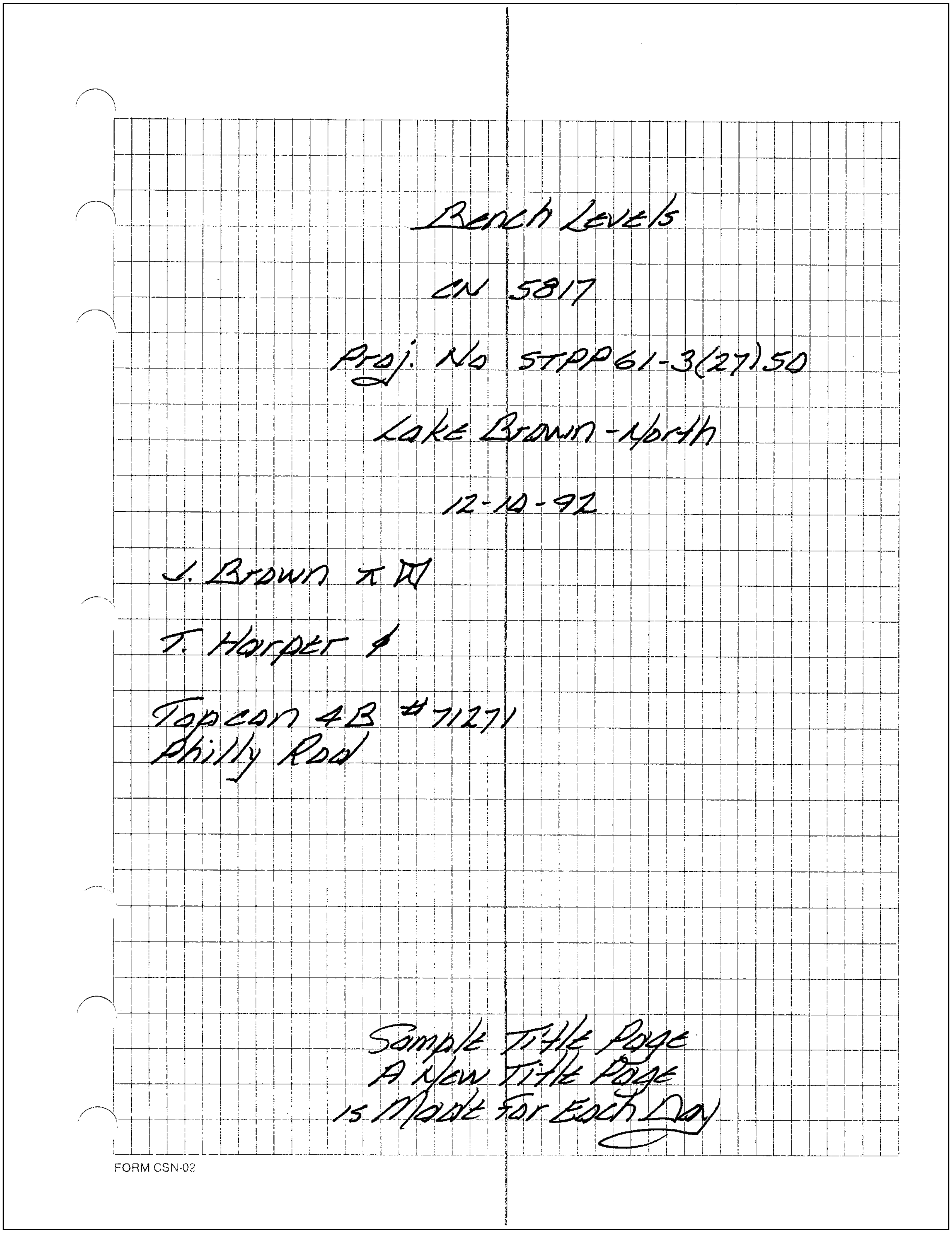
Appendix A
Sample Notes

Sample surveying note keeping formats are presented in this appendix.

Table of Figures



Bench Levels

CN 5817

Proj. No STPP61-3(27)50

Lake Brown-North

12-10-92

J. Brown π ∇

T. Harper ɸ

Topcon 4B #71271
Philly Rod

Sample Title Page
A New Title Page
is Made For Each Day

FORM CSN-02

Figure A-1
Sample Title Page – Bench Levels

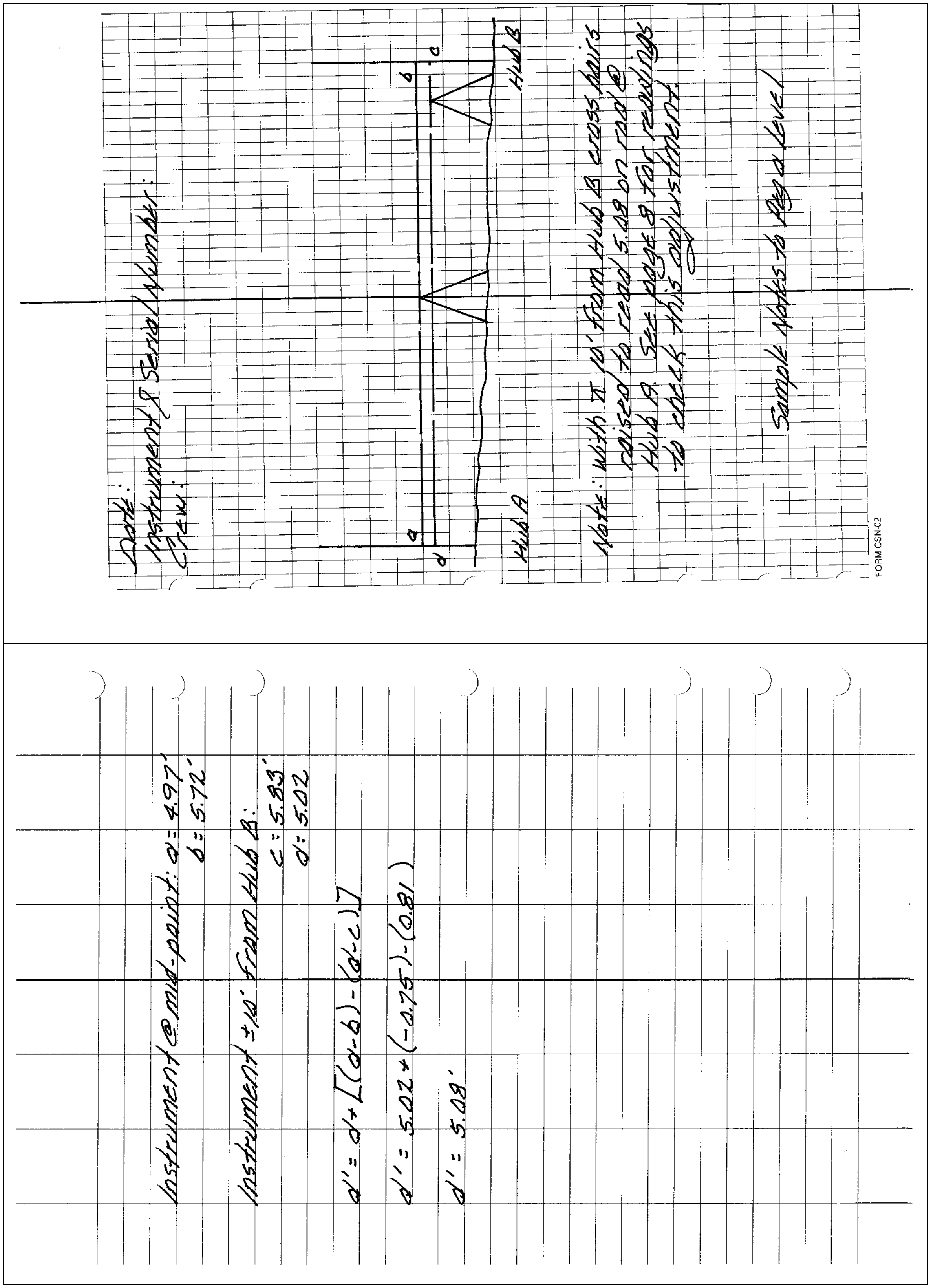


Figure A-2
Peg a Level

STATION	+ BS	HI	− FS	Elev	ADJ Elev	
D 402				3071^{18}	3071^{18}	D402 - STANDARD USC&GS DISK SEE DATA SHEET NAVD88 = 3071^{18}
	7^{22}	3078^{40}				
TP			9^{08}	3069^{32}		
	10^{11}	3079^{43}				
A 4444			1^{18}	3078^{25}	3078^{25}	CONTROL POINT SEE ABSTRACT
	7^{51}	3085^{76}				
TP			2^{83}	3082^{93}		
	10^{96}	3093^{89}				
B 4444			0^{03}	3093^{86}	3093^{87}	CONTROL POINT SEE ABSTRACT
	9^{56}	3103^{42}				
603 4444			8^{02}	3095^{40}	3095^{41}	FND 5/8" REBAR W/2" A.C. FLUSH W/GROUND CAP STAMPED "6034444 2004" SEE CONTROL TRAVERSE NOTES FOR LOCATION
	4^{02}	3099^{42}				
TP			6^{37}	3093^{05}		
	5^{77}	3098^{82}				
TP			4^{32}	3094^{50}		
	10^{21}	3104^{71}				
C 4444			9^{17}	3095^{54}	3095^{56}	CONTROL POINT SEE ABSTRACT
	8^{83}	3104^{37}				
TP			7^{36}	3097^{01}		
	5^{46}	3102^{47}				
D 4444			0^{19}	3102^{28}	3102^{30}	CONTROL POINT SEE ABSTRACT
	6^{37}	3108^{65}				
E 4444			1^{27}	3107^{38}	3107^{40}	ERROR = −0.03
	8^{76}	3116^{14}				
TP			3^{14}	3113^{00}		MAX ALLOWABLE ERROR = $3 \cdot 0.005 \sqrt{\# FS}$ = $0.015 \sqrt{14} = 0.06$
	1^{17}	3114^{17}				
TP			12^{82}	3101^{35}		
	5^{56}	3106^{91}				
D 403			11^{87}	3095^{04}	3095^{07}	SEE DATA SHEET NAVD88 = 3095^{07} BENCH LEVELS
	Σ=+101^{51}		Σ=−77^{65}			

FORM CSN-02

Figure A-3
Bench Levels

STA	+ B.S.	— HI	FS	Elev	ADJ. Elev	
A2900				3117.52	3117.52	CONTROL POINT SEE ABSTRACT
	7.17	3124.69				
			8.72	3115.97		
	11.82	3127.79				
B2900			1.96	3125.83	3125.83	CONTROL POINT SEE ABSTRACT
	8.18	3134.01				
			2.97	3131.04		
	10.82	3141.86				
			4.53	3137.33		
	5.86	3143.19				
C2900			2.02	3141.17	3141.16	CONTROL POINT SEE ABSTRACT
	3.17	3144.34				
D2900			10.14	3134.20	3134.19	CONTROL POINT SEE ABSTRACT
	5.11	3139.31				
			7.82	3131.49		
	2.77	3134.26				
			9.42	3124.84		
	5.01	3129.85				
E2900			9.72	3120.13	3120.11	CONTROL POINT SEE ABSTRACT
	3.27	3123.40				
			9.14	3114.26		ALLOWABLE ERROR = ± 0.05
	5.62	3119.88				ACTUAL ERROR = + 0.02
			10.72	3109.16		
	10.08	3119.24				
F2900			7.70	3111.54	3111.52	CONTROL POINT SEE ABSTRACT
	Σ=+78.88		Σ=-84.86			

FORM CSN-02

Figure A-4
Differential Levels To Additional Control Points

STATION	B S	H I	F S	Elev	Adj Elev
C1560					3019.40
	10.27	3029.67			
TP			11.82	3017.85	
	5.67	3023.52			
TP			1.18	3022.34	
	9.01	3031.35			
TP			3.14	3028.21	
	8.15	3036.36			
6091560			3.86	3032.50	3032.52
	5.17	3037.67			
TP			8.02	3029.65	
	3.78	3033.48			
TP			5.96	3027.47	
	4.81	3032.28			
TP			8.11	3024.17	
	5.47	3029.64			
D1560			4.27	3025.37	3025.42
	Σ=52.33	Σ=	−46.36		
		+5.97			

Control Point See Abstract
(3019.40)

Target – See Abstract
Note: Cap 0.15 Above Ground
Ground Elev = 3032.37

Control Point See Abstract
(3025.42)

Allowable Error = ± 0.10
Actual Error = − 0.05

Figure A-5
Differential Levels To Photo Control Points

BS	Stad	Check	FS	Stad	Check
8.266			3.491		
	0.161			0.171	
8.105		-.0013	3.320		+.0010
	0.165			0.168	
7.940		8.1037	3.152		3.3210
24.311		8.1037	9.963		3.3210
0.326	0.326		0.339	0.339	
6.574			4.623		
	0.216			0.204	
6.358		-.0003	4.419		+.0013
	0.217			0.200	
6.141		6.3577	4.219		4.4203
19.073		6.3577	13.261		4.4203
0.433	0.433		0.404	0.404	
(1)					
	(4)				
(2)		(6)			
	(5)				
(3)		(7)			
(8)		(10)			
(9)	(11)				

Checks

(4) = (1) - (2), (5) = (2) - (3), (6) = (4-5) ÷ 3

(7) = (2) + (6), (8) = (1) + (2) + (3)

9 = (1) - (3), (10) = (8) ÷ 3, (11) = (4) + (5)

(7) must = (10) ; (9) must = (11)

Elev	Sta	Sum
3098.461	BM29	
+8.104		+326
3106.565	HI	
- 3.321		-339
3103.244	TP1	-13
+6.358		+433
3109.602	HI	
- 4.420		-404
3105.182	TP2	+16
		(12)

(12) = Cumulative distances

Note: (4) ; (5) must equal to 0.005 or less
Three Wire Level

FORM CSN-02

Figure A-6
Three-Wire-Levels

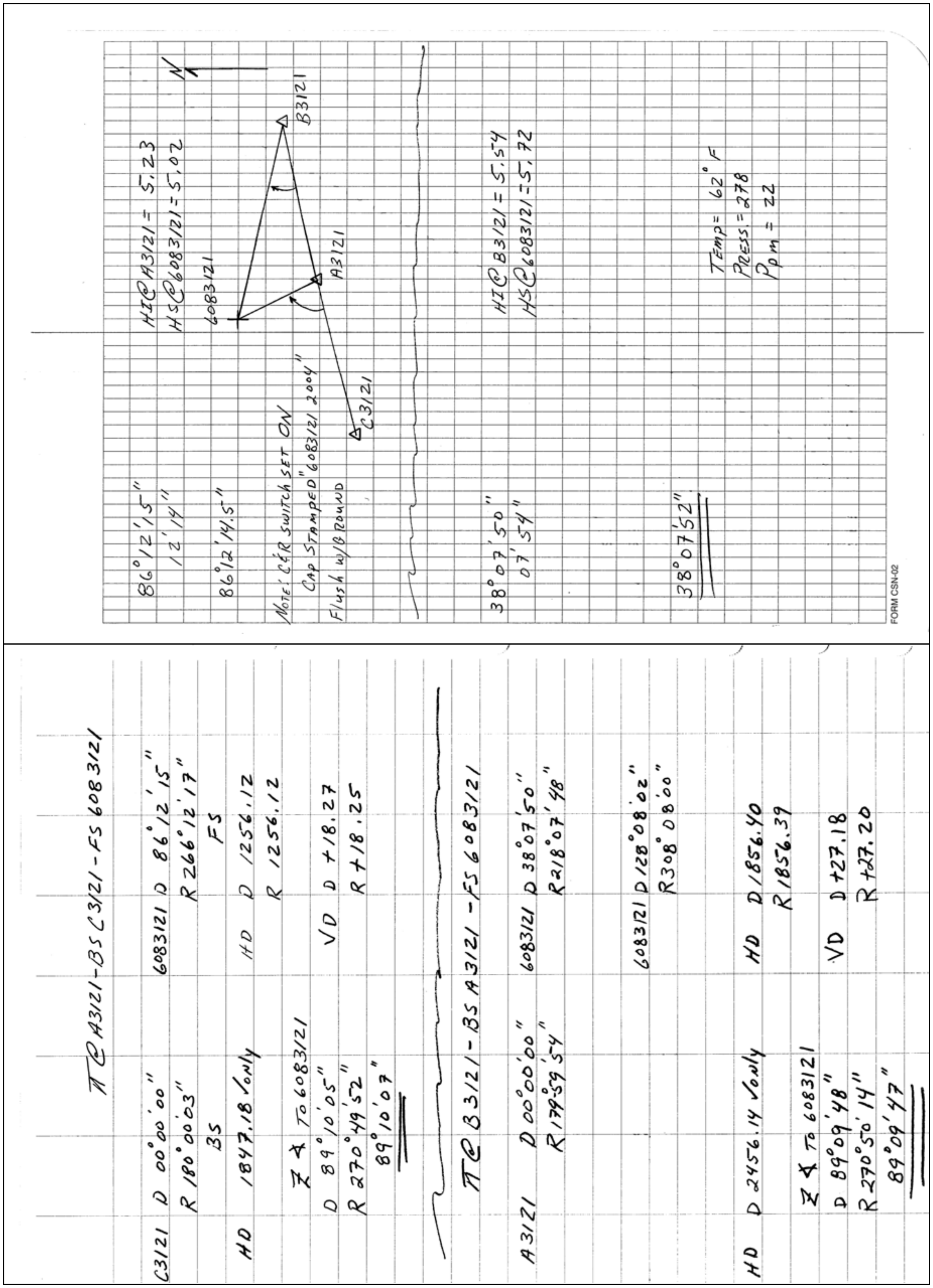


Figure A-7
Trig-Levels – Total Station

Control Traverse

CN 5817

Proj. No. STPP61-3(27)50

Lake Brown - North

12-17-92

J. Brown ⊼ ☒

T. Harper

Lietz Set II B # 20958

All prisms - 30 mm offset

Sample Title Page
A New Title Page
is Made for Each Day

FORM CSN-02

Figure A-8
Sample Title Page – Control Traverse

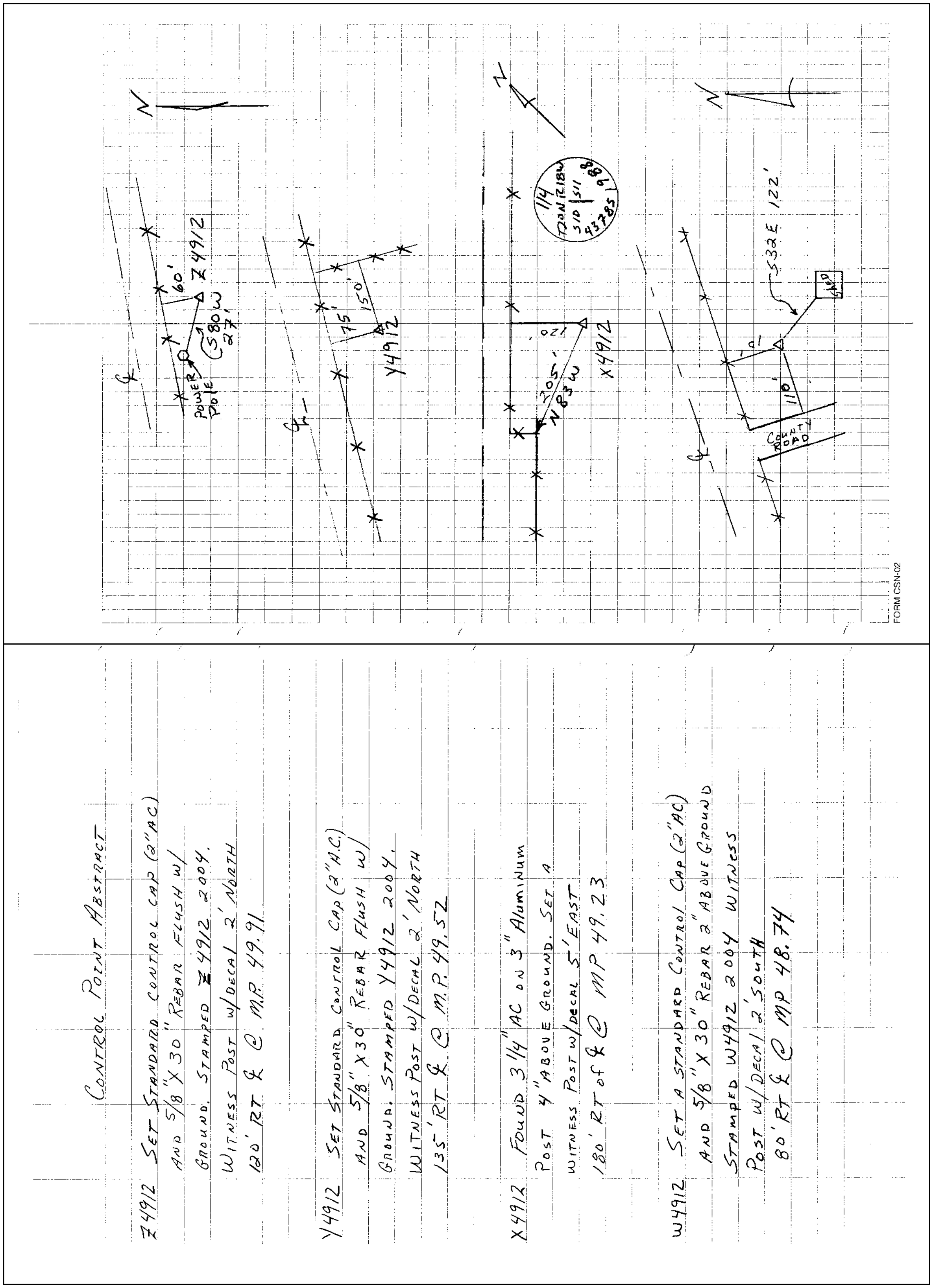


Figure A-9
Control Point Abstract

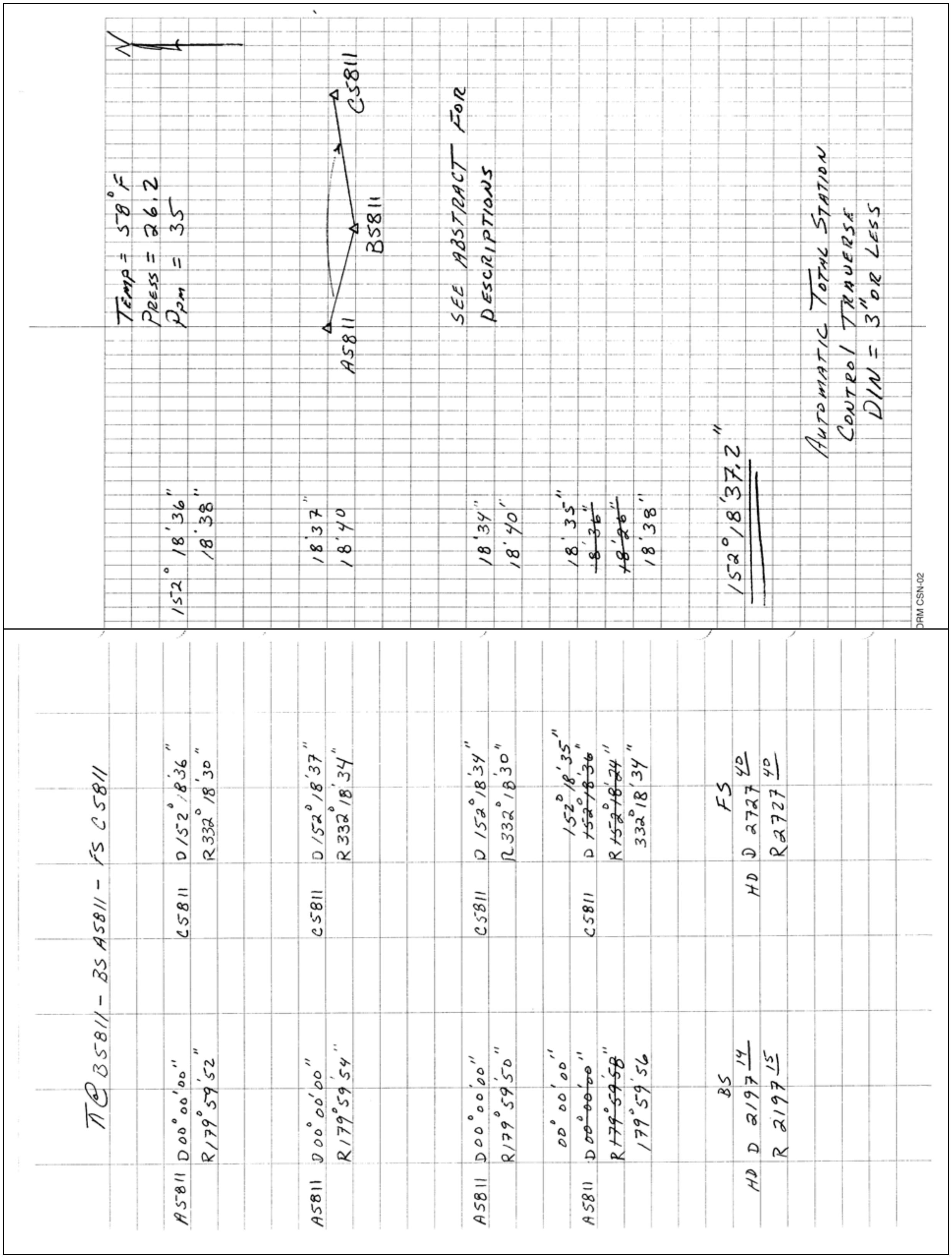


Figure A-10
Control Traverse – Total Station

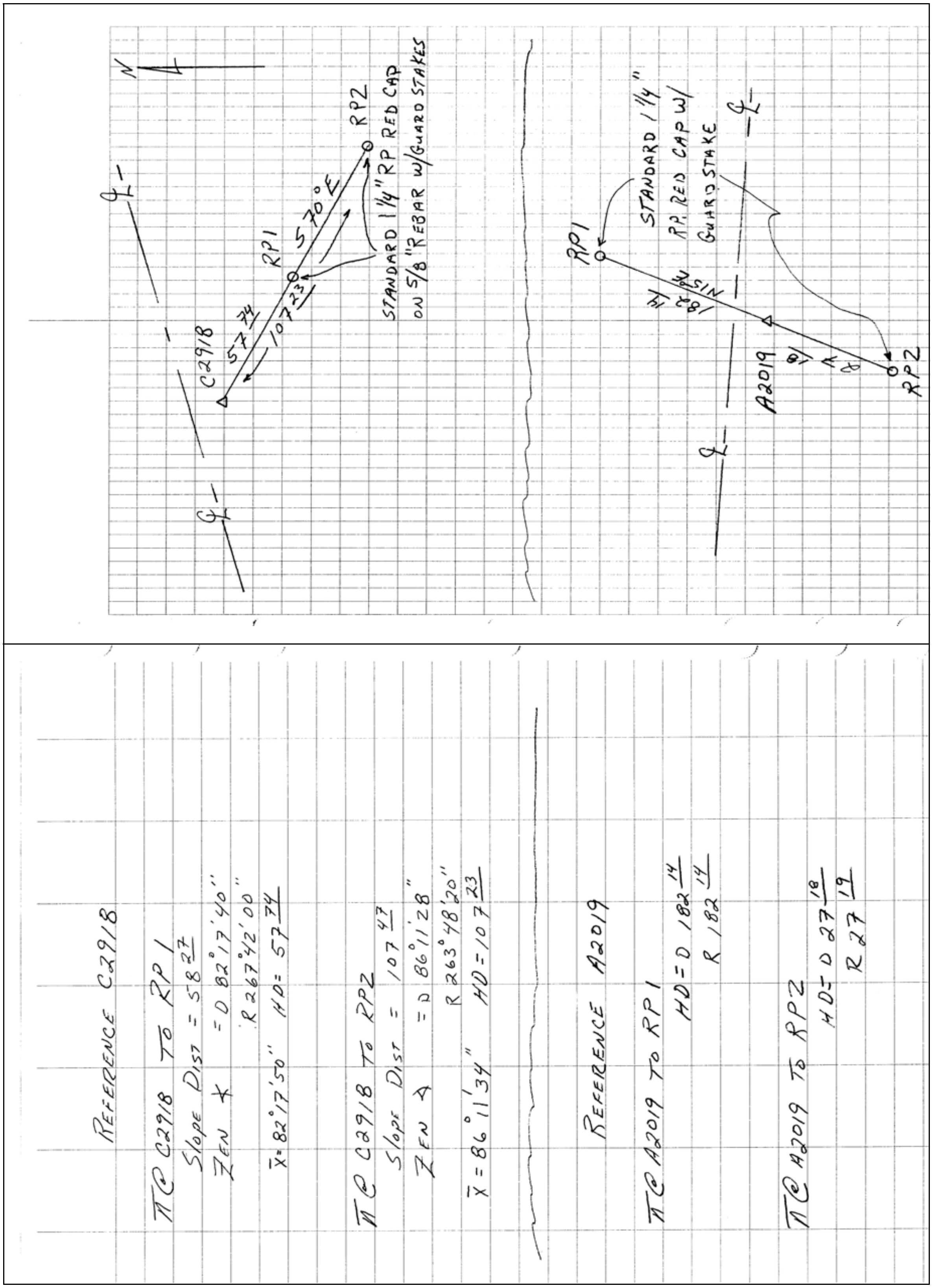


Figure A-11
Reference Points – Total Station

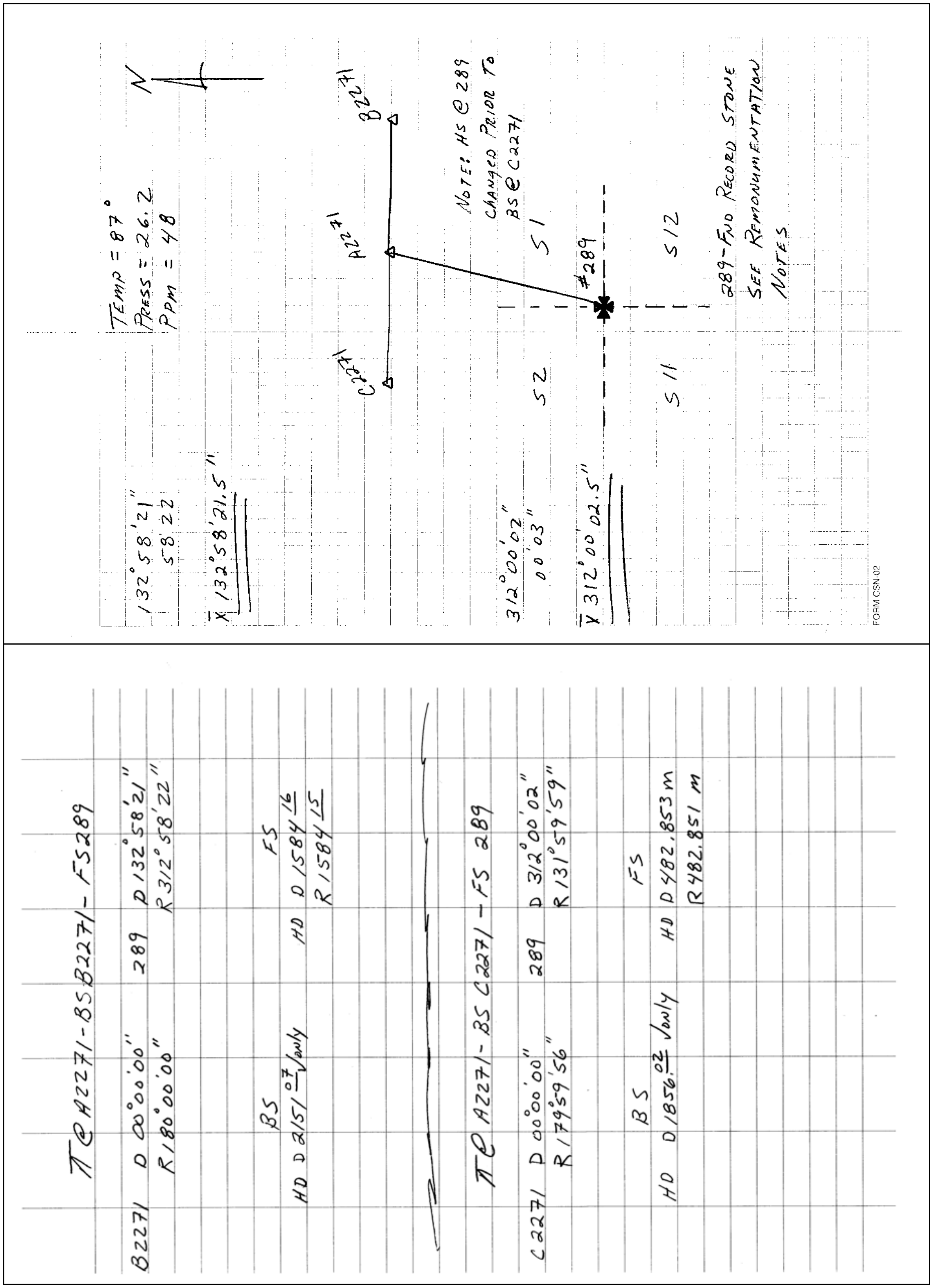


Figure A-12
Tie From One Control Point – Total Station

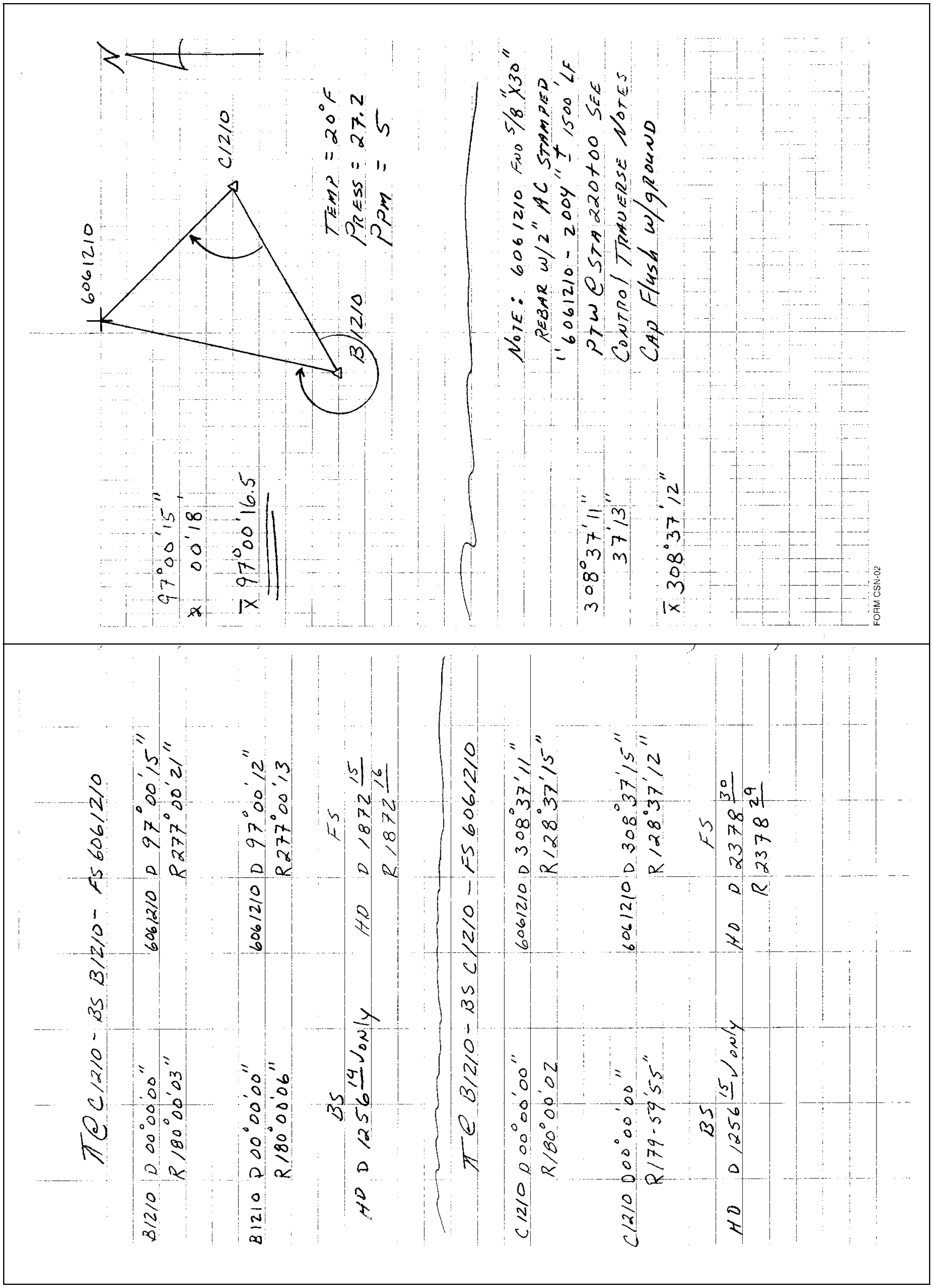


Figure A-13
Tie From Two Control Points – Total Station

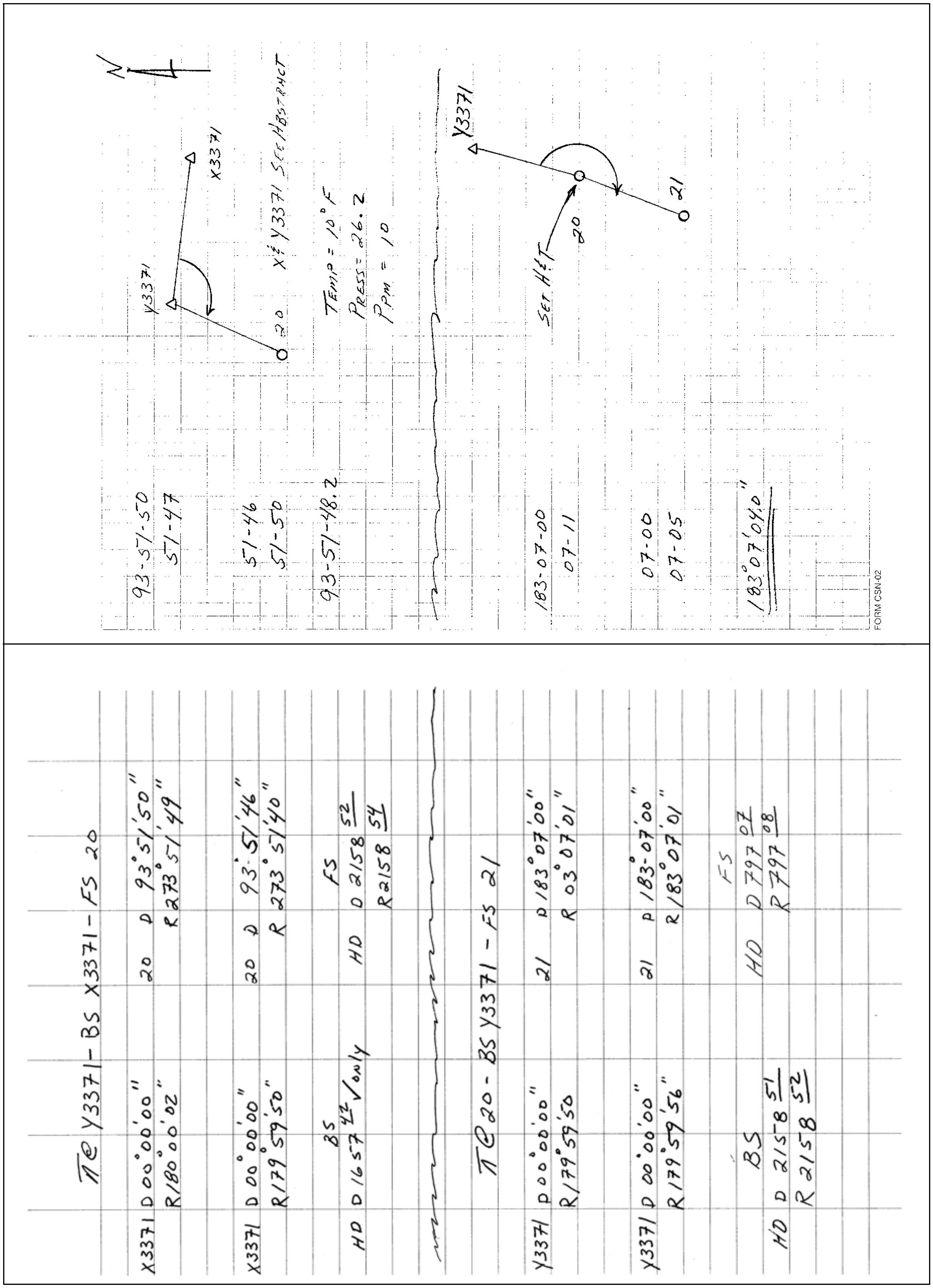


Figure A-14
Beginning a Traverse to Controlling Property Corner – Total Station

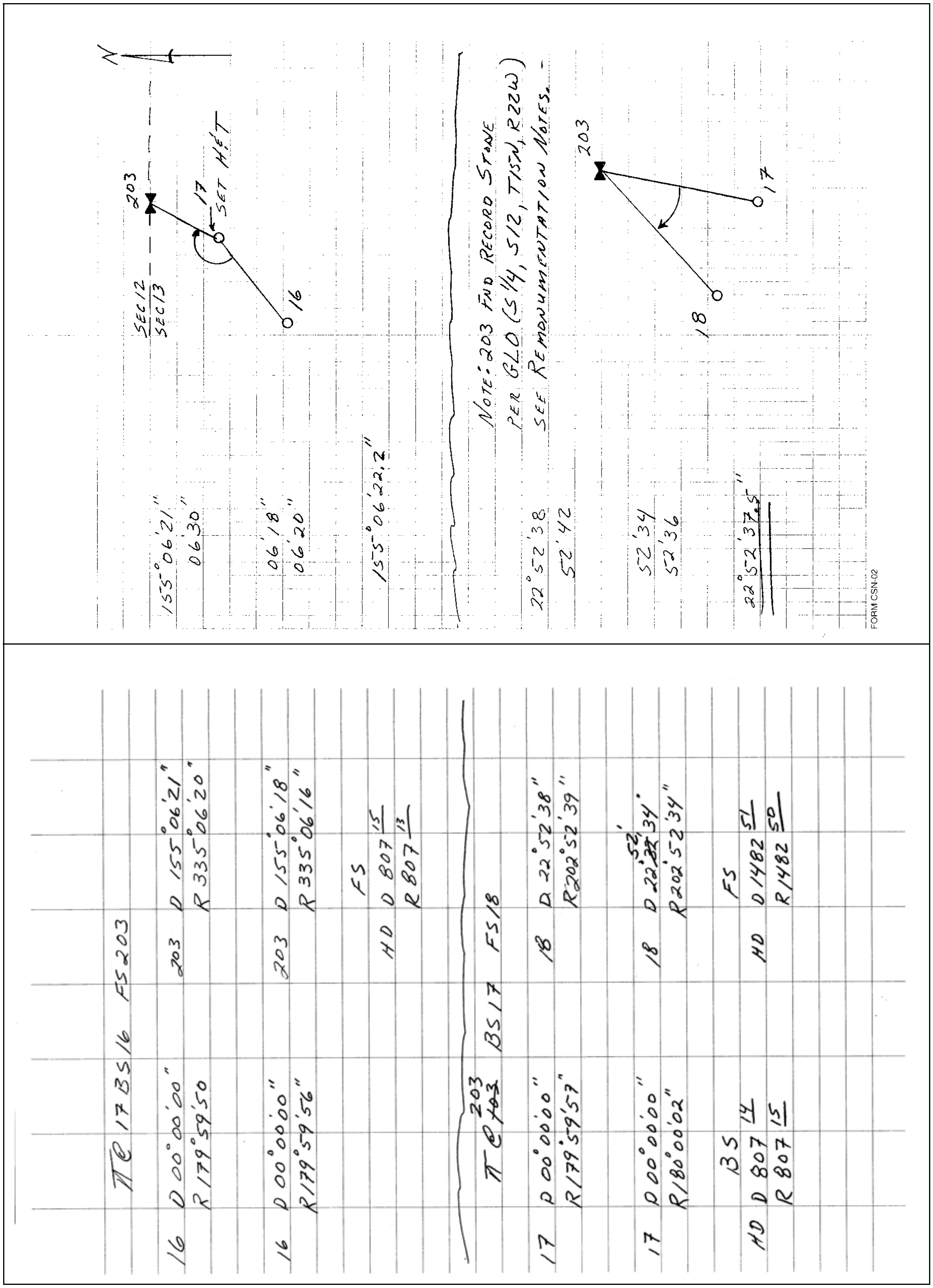


Figure A-15
Traverse to a Controlling Property Corner – Total Station

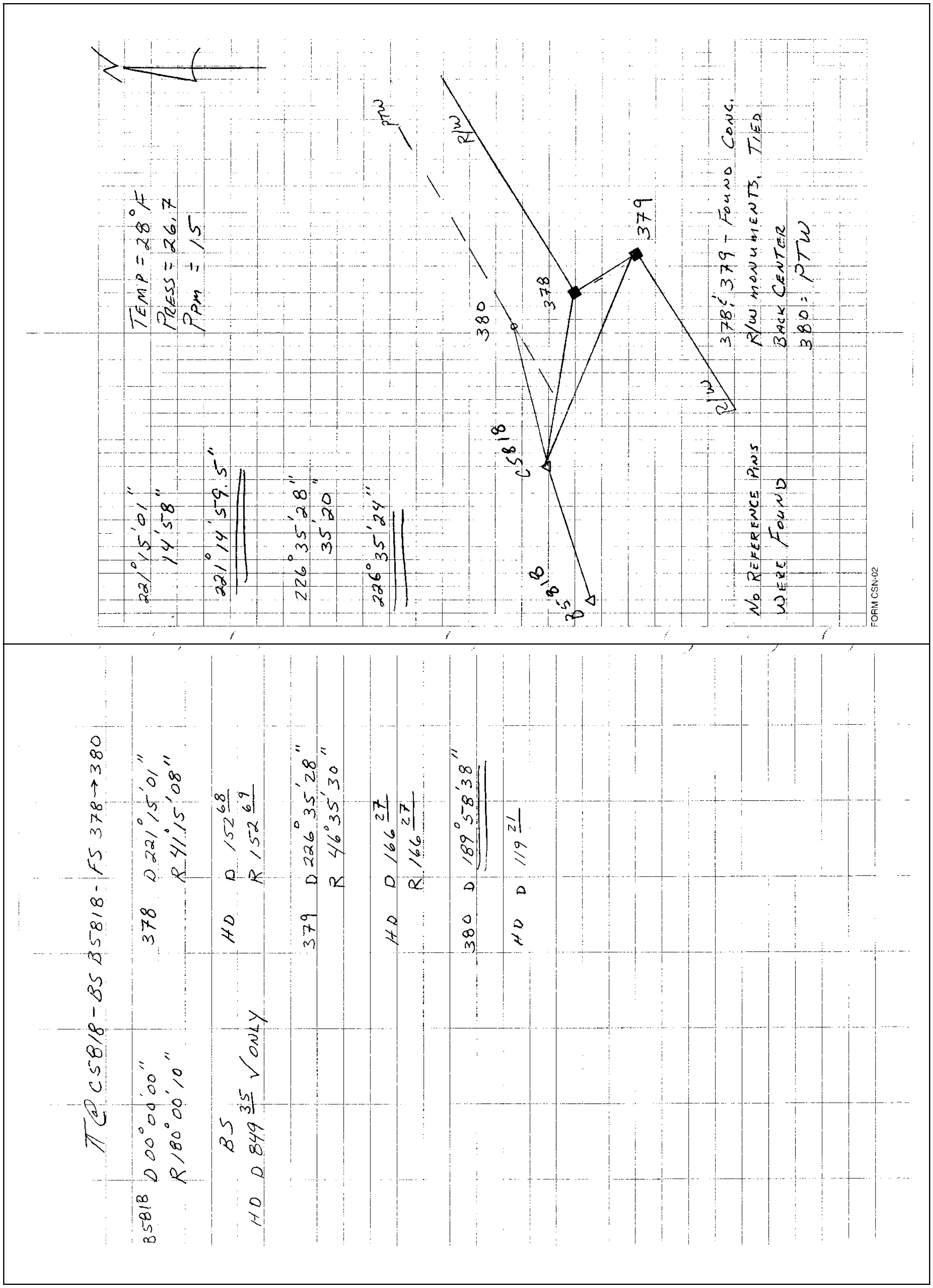


Figure A-16
Side Shots to R/W and PTW – Total Station

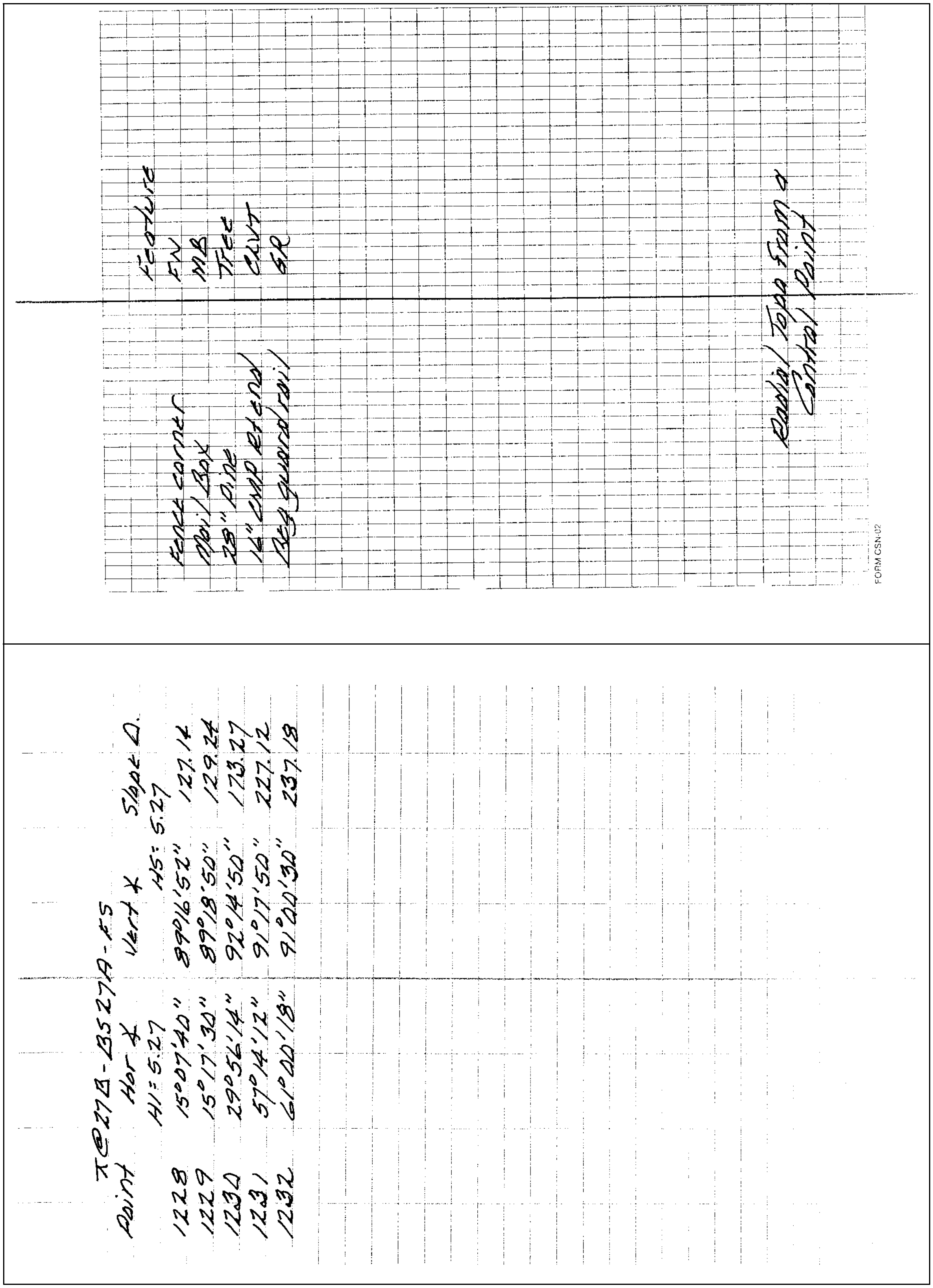


Figure A-17
Radial Topography From a Control Point

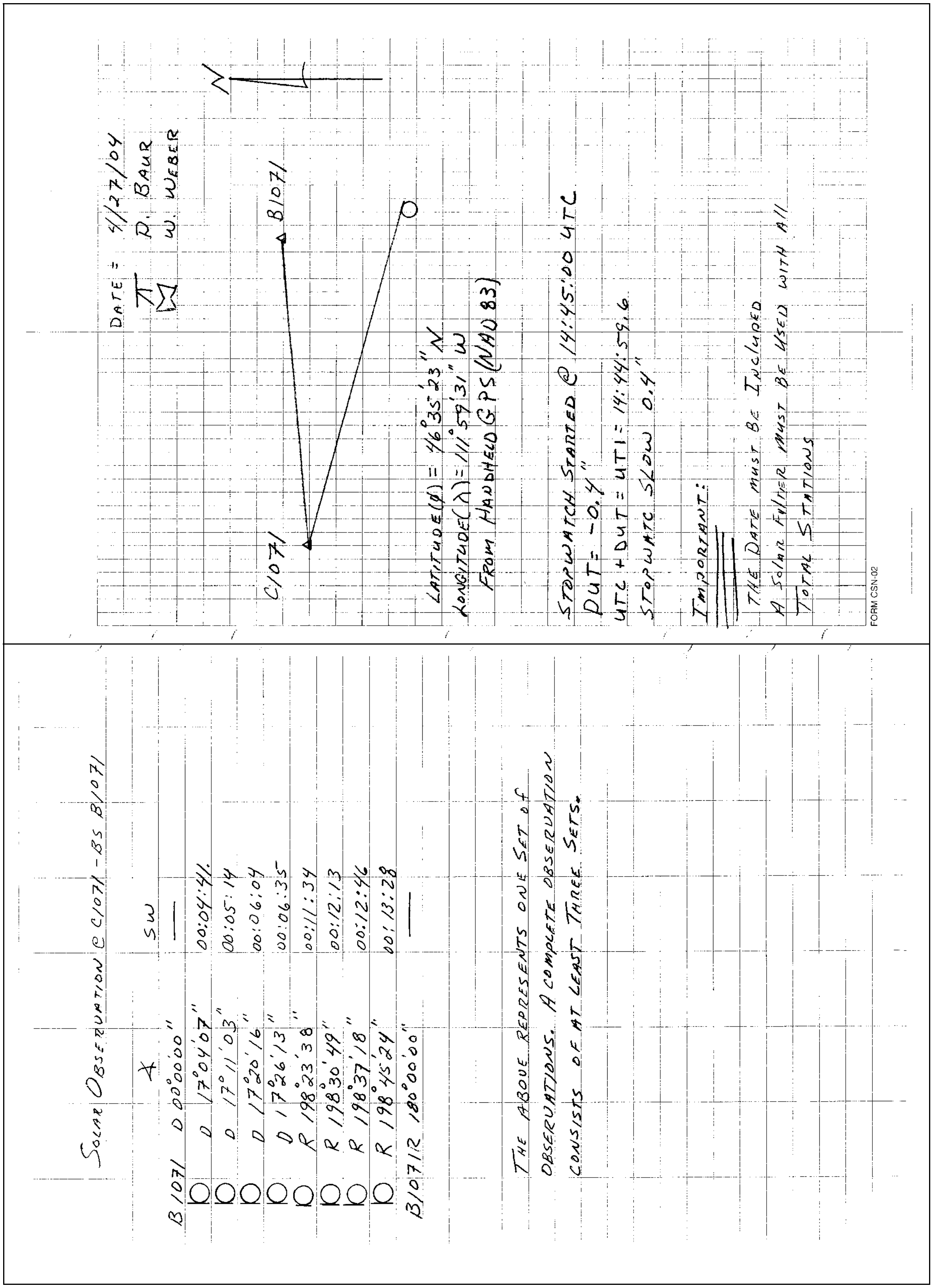


Figure A-18
Solar Observation

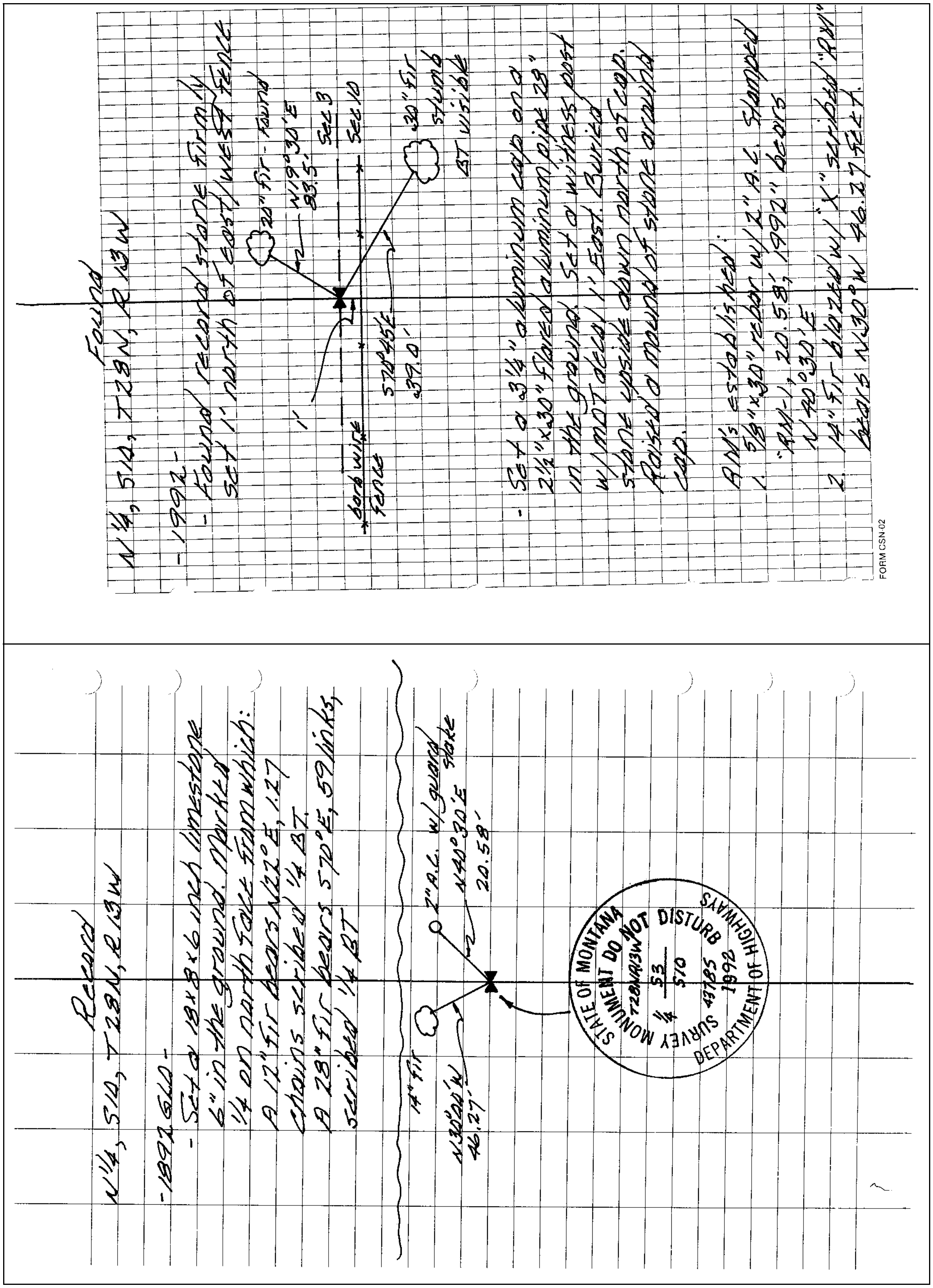


Figure A-19
Monument Notes – Sample 1

Record

N 1/4, S 30, T 3S, R 22 W

- 1879 GLO -
- Set a 18×12×6 inch sandstone 12 inches in ground. Marked 1/4 on north face. Pits impractical

Found

N 1/4, S 30, T 2S, R 22 W

- 1992 -
- Searched for not found (SFNF) Scaled distance from highway to corner using quad map - 1020'. Paced into area using compass.

Searched ± 1 1/2 hours. Set a lath marked "Approx 1/4" @ paced distance. Lath is in range land. No evidence suggests stone destroyed.

Should be search for again after ties made to section corner to the east.

FORM CSN-02

Figure A-20
Monument Notes – Sample 2

Record

Southwest corner S16, T15N, R43E

- 1889 GLO -
  - Set a 22x16x8 inch quartzite stone 16 inches in ground. Marked w/ 3 grooves on south and 4 grooves on east. Dug pits 18 inches square - 12" deep in each section @ 5½ feet distant.

Found

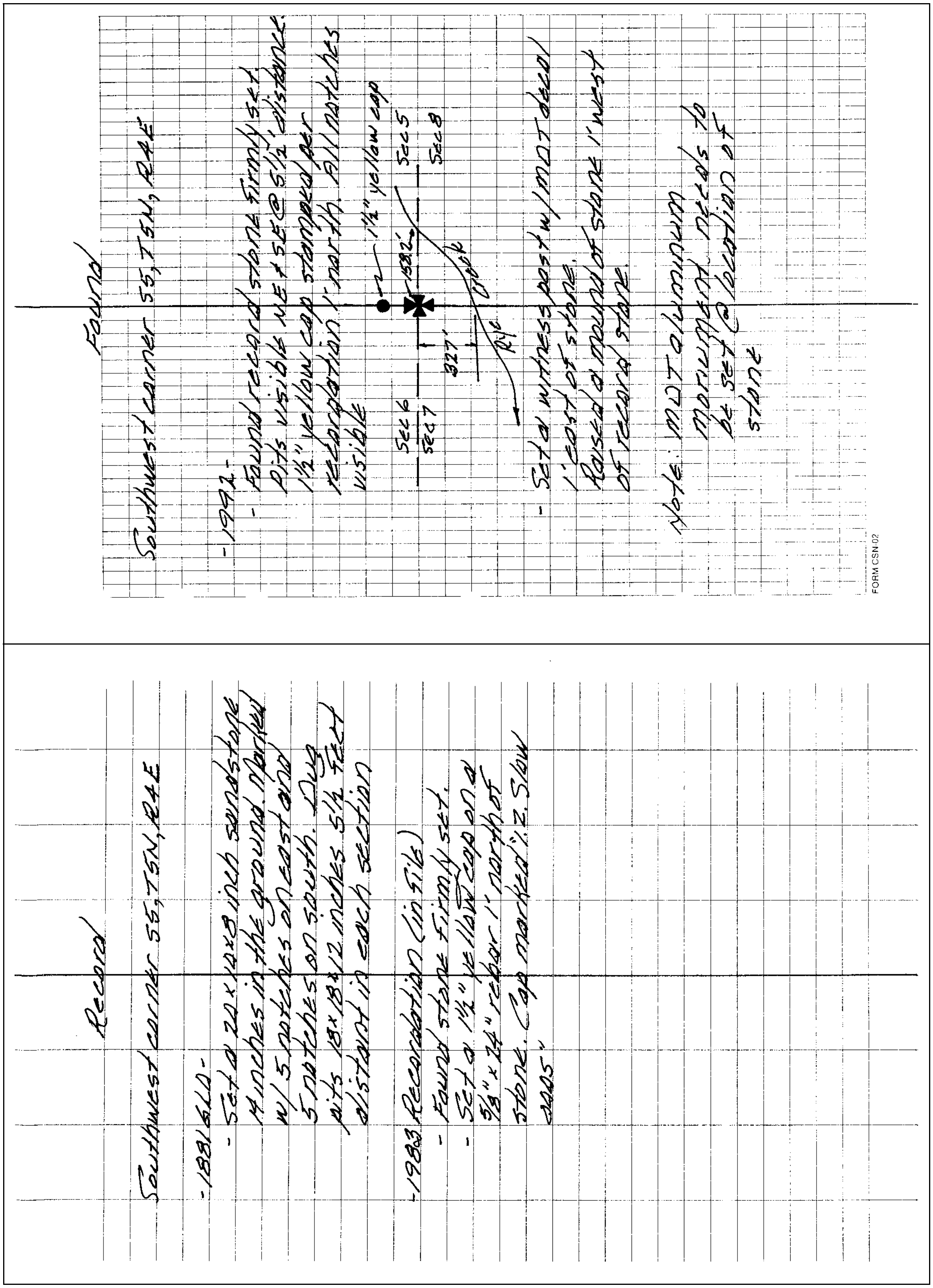
Southwest corner S16, T15N, R43E

- 1992 -
  - SFNF
    Corner falls in wheat field. Mr. James Walker who farms the land was born-house located in SW¼ of Sec 16 in 1921. Grandfather homesteaded Sec 16 & 17. Father farmed the homestead. Mr. J. Walker doesn't know anything about this corner nor the S¼ corner of Sec 16. He doesn't recall his Grandfather or Father telling him anything either.

    Corner is probably lost.

FORM CSN-02

Figure A-21
Monument Notes – Sample 3



Record

Southwest corner S5, T5N, R4E

-1881 GLO-

- Set a 22×14×8 inch sandstone 14 inches in the ground. Marked w/ 5 notches on east and 5 notches on south. Dug pits 18×18×12 inches 5½ feet distant in each section

-1983 Recordation (in file)

- Found stone firmly set.
- Set a 1½" yellow cap on a 5/8"×24" rebar 1' north of stone. Cap marked "I.Z. Slow 2005"

Found

Southwest corner S5, T5N, R4E

-1992-

- Found record stone firmly set. Pits visible NE & SE @ 5½' distance. 1½" yellow cap stamped per recordation 1' north. All notches visible

- Set a witness post w/ MDT decal 1' east of stone. Raised a mound of stone 1' west of record stone.

Note: MDT aluminum monument needs to be set @ location of stone

FORM CSN-02

Figure A-22
Monument Notes – Sample 4

Report on Condition of Survey Mark

Station Name: C 755

State: OR County: Douglas

Agency Disk: USC&GS X NGS USGS Other:_______

Station Recovered By: Curt Smith

Organization: National Geodetic Survey

Address: P.O. Box 12114 Salem, OR 97309

Telephone: 503-986-3543 Date of Recovery: 11/7/98

Stamping on the Disk:________

(Example: *Y 126 RESET 1982* or *JONES 1986* or *JONES NO 2 1986*)

Recovery Condition: **Good** **Poor** **Not Found** **Destroyed**

(Note: **Circle one**. Destroyed means you recovered disk and are returning it to address listed below)

Explanation of Recovery: Highway 13 widening project. Destroyed bench mark after it was referenced to C755 RESET

Changes (if any) to the Station Description: Note - Destroyed - Disk recovered

(Note: *RAD* = Recovered As Described; Changes example: *TO REACH THE STATION FROM THE INTERSECTION OF U.S. HIGHWAY 36 AND STATE HIGHWAY 24 IN SALEM, GO SOUTH ON U.S. HIGHWAY 36 FOR 3.5 KILOMETERS TO THE MARK ON THE LEFT. THE MARK IS 34.5 M EAST FROM THE CENTER OF THE HIGHWAY, 15.4 M SOUTH FROM A POWER POLE.* Try to give four measured ties in the description. If only a witness post or other feature is added, report only that information: *THE MARK IS 2.5 M EAST FROM A FIBERGLASS WITNESS POST.*

Return this form to: Curt Smith
National Geodetic Survey
P.O. Box 140533
Boise, ID 83714
Telephone: 208-332-7197

Include the following information if known:

NGS database PID: OA0686 USGS Quad: Roseburg

Latitude: N 43° 43' 12" Longitude: W 124° 06' 13"

Elevation: 6.663 m Setting Type: Concrete Post

Figure A-23
Report on Condition of Survey Mark

Report on Relocation of Reset Bench Mark

Station Designation: C 755 Reset	Level Line Number:	State: Oregon	County: Douglas	
Latitude: 43° 43′12″ N	Longitude: 124° 06′ 13″ W		Position Accuracy: +/- ___ Scaled GPS Other:	
Project Name: Highway 13 Widening		Highway Name: State Highway 13		Key#:

Information About Old Mark (circle or check options):

Exact Stamping of Old Disk: C 755 1988

Agency Pre-Cast in Disk/Monument Cover: US Coast + Geodetic Survey

Published Elevation of Old Bench Mark: 6.663 Meters Feet Datum: NAVD 88

Old description agrees as found? Very well More or less Poorly Not at all

Old monument solidly in ground? Yes No, explain: ___

Any damage to disk or monument? No Yes, explain: ___

Anticipated date old mark to be Disturbed or Destroyed 11/98

Describe reason for reset: Highway 13 widening destroyed after it was referenced to C 755 RESET

Information About New Mark:

Exact Stamping of New Disk: C 755 RESET 1998 Date Set: 8/8/98

Agency Pre-Cast in Disk/Monument Cover: NATIONAL GEODETIC SURVEY

Type of Disk Set: Vertical Control Magnetic Material: Rebar in Concrete Monument

Site suitable for use with GPS geodetic surveying (e.g., few obstructions to satellites) Yes No Don't know

Setting Classification of New Monument (circle monument type 1, 2 or 3; circle or check options):

1. Concrete Post:
   a. Diameter of Monument: 0.35 m Depth of Monument: 1.2 m
   b. Top of Monument: Flush X Projecting ☐ Recessed ☐ ___ m, with ground.
2. Disk Set in Drill Hole:
   a. Rock Outcrop ☐ or Boulder ☐ Approximate exposure: ___ m by ___ m
   b. Bridge Abutment ☐ or Other, explain: ___
   c. Mark relationship with surface: Flush ☐ Projecting ☐ Recessed ☐ ___ m, with ___
3. Rod Mark Driven to Refusal:
   a. Depth of rod driven: ___ m To refusal, Slow time met Grease filled sleeve depth: ___ m
   b. Top of rod recessed ___ cm below monument cover.
   c. Top of monument cover: Flush Projecting Recessed m, with

Reported By: Date: 8/8/98

Agency: National Geodetic Survey Contact: Curt Smith

Address: P.O. Box 140533 Telephone: (208) 332-7197

City / State / Zip: Boise, Idaho 83714 Fax: ( ) ___

E-mail:

Figure A-24
Report on Relocation of Reset Bench Mark

Station Description

New Station Designation: C 755 RESET

Description of Physical Location

The station is located about 3.5 miles southeast of Roseburg, on north Right-of-Way of State Highway 13, at highway milepost 6.3. In top of a round concrete monument

To reach the station from The junction of interstate highway 5 and State highway 13 (exit 210). In Roseburg, go southeast on highway 13 for 3.9 miles to the station on the left just before reaching the private drive leading northeast to a residence.

Permanent Station Reference Objects

Distance to Mark	Direction to Mark	Description of Reference Objects
32.3 m	Northeast	From the center of Highway 13
28.6 m	Northwest	From the center of Private drive
3.3 m	Southeast	From power pole number A277361
1.1 m	Southwest	From Right-of-way Fence
1.0 m	Northwest	From a fiberglass witness post

Figure A-25
Station Description

Observations for Relocation of Bench Mark

Original Mark Stamping: C 755 1988
PID (if known): OA∅686
Elevation: 6.663 (ft / m)
Vertical Datum: NGVD 29 or NAVD 88

Replacement Mark Stamping: C 755 RESET 1998
Date of Leveling: 8/8/98
Computed Elevation: (from below) 3.391 (ft / m)

State: OR County: Douglas Latitude: N 43° 43′ 12″ Longitude: W 124° 06′ 13″ Hz. Datum: NAD 83

Level Instrument: Manufacturer: Wild Model Number: NA 3003 Serial Number: 95-1926
Rod #1: Manufacturer: Wild - 3 piece Invar Model Number: Serial Number: Rod Scale Units: m
Rod # 2 (optional): Model Number: Serial Number: Rod Scale Units:

Forward Running: Old to New (ft / m) Observer: Curt Smith Rodman # 1: Rick Tallman Rodman # 2: Jim Short

Start Time: Temperature: 73 F C Sun Conditions: clear Wind Conditions: Breezy Stop Time:

Point	Backsight	H.I.	Foresight	Elevation	Distance	Remarks
C 755			Starting or Published Elevation =	6.663		
	0.47760	7.14000				
TP 1			1.63280	5.5072		
	1.28673	6.79393				
TP 2			2.01611	4.77782		
	1.46246	6.24028				
C 755 Reset			2.84936	3.39092		
	Forward Running Elevation / Total Distance =					

Backward Running: New to Old (ft / m) Observer: Curt Smith Rodman # 1: Rick Tallman Rodman # 2: Jim Short

Start Time: Temperature: 73 F C Sun Conditions: Clear Wind Conditions: Breezy Stop Time:

Point	Backsight	H.I.	Foresight	Elevation	Distance	Remarks
			Starting Elevation from Forward Running Elevation =	3.39092		
C 755 Reset	2.84177	6.23269				
TP 2			1.45466	4.77803		
	2.00920	6.78723				
TP 1			1.27970	5.50753		
	1.63636	7.14389				
C 755			0.48100	6.66289		6.663 (published)
						- 6.66289
						0.00011 ✓
	Backward Running Elevation / Total Distance =					

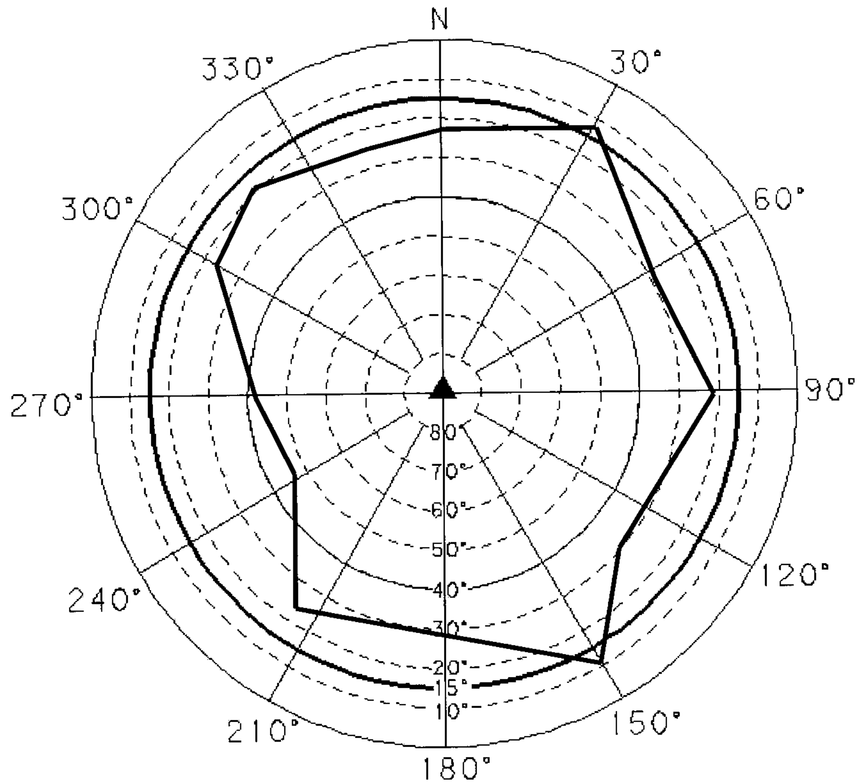
Forward Starting or Published Elevation =			
Backward Running Elevation =			
Difference = Forward Starting or Published Elevation – Backward Running =			
Final New Elevation = Forward Running Elevation – (*Difference ÷ 2) *Maintain Sign of Difference =			

Agency / Firm: Oregon Depart. of Trans Submitted By: Curt Smith
Address: 200 Hawthorne Ave, SE B-250 Telephone: (503) 986-3543
City / State / Zip: Salem, OR 97301 E-mail: c.s@state.us

Figure A-26
Observations For Relocation Of Bench Mark

#	HORZ ANGLE	VERT ANGLE
1	0°	25
2	30	15
3	60	30
4	90	20
5	105	10
6	130	30
7	150	12
8	215	25
9	240	50
10	270	40
11	300	20
12	320	18
13	345	25
14		
15		
16		
17		
18		
19		
20		

STATION VISIBILITY DIAGRAM



INSTRUCTIONS;

Identify obstructions by azimuth and elevation angle (above horizon) as seen from Station mark. Include distance and direction to nearby structures and reflective surfaces (potential multipath sources).

STATION I.D. C4401 CONTROL NUMBER 4401

DATE 9/16/2004

DECLINATION (IF ANY) SET IN COMPASS NONE
NAME JOHN SMITH DISTRICT BUTTE

COMMENTS

DECLINATION INFORMATION IS REQUIRED

Figure A-27
Station Visibility Diagram

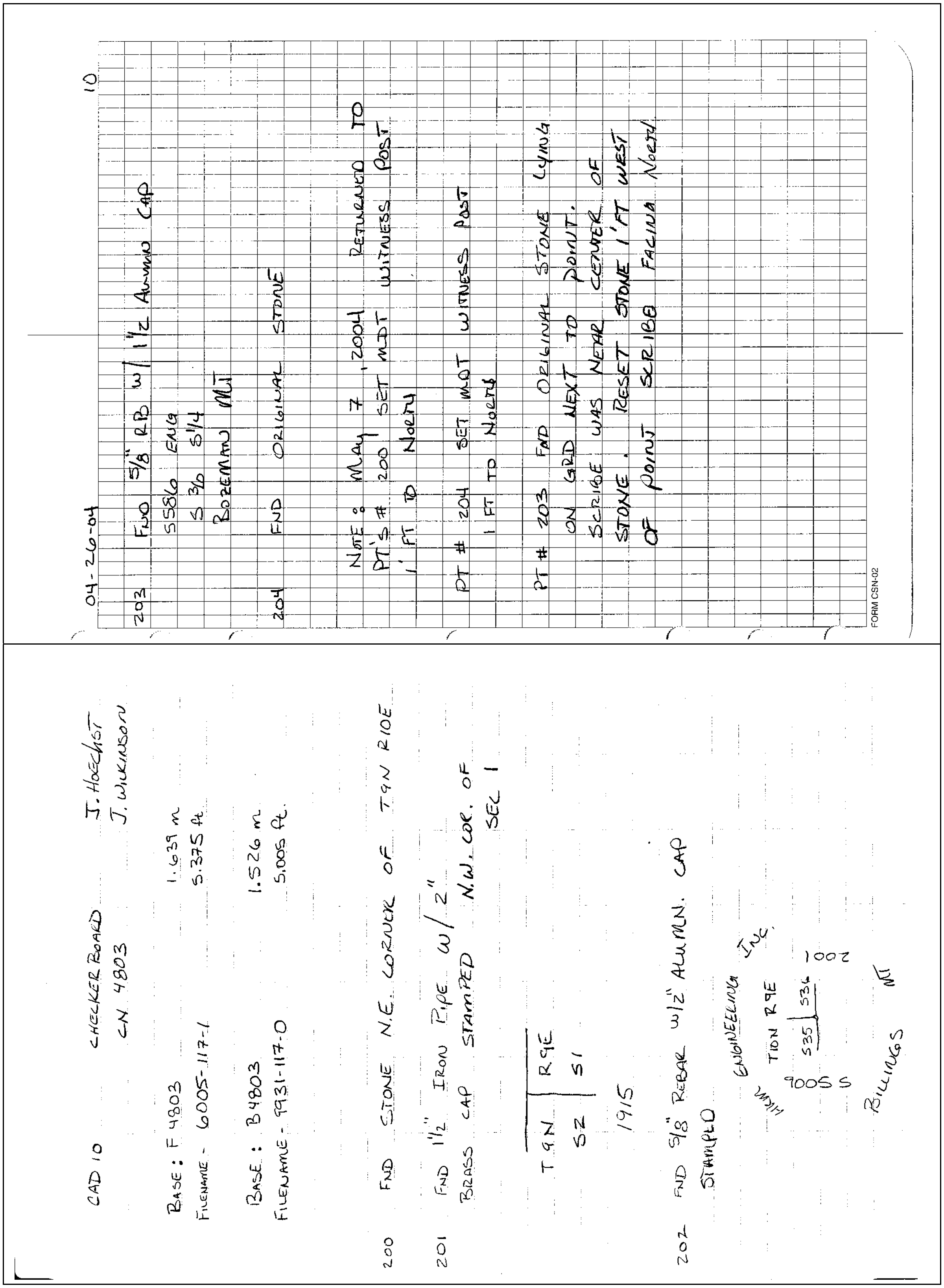


Figure A-28
RTK Field Notes

STATIC/FAST STATIC LOG
*** FILL IN ALL INFORMATION COMPLETLY ***

CONTROL & PROJECT NUMBER : 4830 Im 94-3(56)88 DATE: 8/24/04

RECEIVER NUMBER 62 FILE NAME RV62237-0

LOCATION (PROJECT NAME) D4-INTERSTATE STR REHAB

OBSERVER SMITH

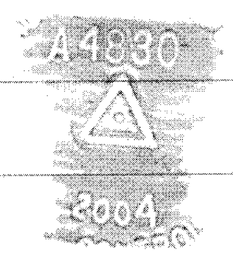
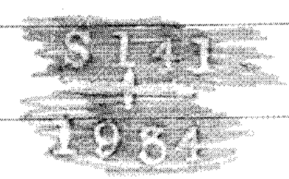
RECEIVER MODEL: (CHECK CORRECT MODEL)

☐ 4000 ☐ 4700 ☒ 5700 ☐ OTHER (SPECIFY)

ANTENNA TYPE: (CHECK CORRECT ANTENNA TYPE)

☐ COMPACT L1/L2 W/ GP ☐ MICROCENTERED L2/L2 W/GP ☒ ZEPHYER GEODETIC

☐ OTHER (SPECIFY)

GPS STATION NAME	SLANT ANTENNA HEIGHT (M)	SLANT ANTENNA HEIGHT (FT)	LOCAL TIME START/STOP	RUBBING AND COMMENTS
A4830	1.311 1.312 1.313 1.312	4 30	8:30/9:00	
A4830	1.558 1.560 1.559 1.559	5 11	9:05/9:35	
			/	
S141	1.411 1.410 1.411 1.411	4 62	10:05/10:35	
S141	1.430 1.429 1.429 1.429	4 69	10:40/11:10	
			/	
			/	
			/	

MONTANA DEPARTMENT OF TRANSPORTATION

SHEET 1 OF 2

Figure A-29
Static/Fast Static Log

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Appendix B
References

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Appendix C
Glossary

The Glossary includes definitions, abbreviations and surveyor's measures and conversions.

Table of Contents

Section **Page**

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Definitions

A

Abstract A summary of the important features associated with survey points (marks). Information in the abstract includes if the monument was set or found, type of monument including stamping, location relative to the PTW and other features such as fences, and if required a how to reach.

Accessories to corners (USPLS) Physical objects adjacent to corners to which the corners are referred for their future identification or restoration. Accessories include bearing trees, mounds, pits, ledges, rocks, and other natural features to which distances or directions from the corner or monument are known. The accessories are actually a part of the monuments.

Accuracy The degree of conformity with a standard, or the degree of perfection attained in a measurement. Accuracy relates to the quality of a result, and is distinguished from precision, which relates to the quality of the operation by which the result is obtained.

Adjusted position An adjusted value for the horizontal or vertical position of a survey mark (point), in which discrepancies due to random errors in the observed data are distributed according to a predefined mathematical relationship.

Aerial photograph Any photography taken from the air — also called aerial photo.

Aerial photography The art, science, or process of taking aerial photographs.

Aerial survey A survey utilizing photographic, electronic, or other data obtained from an airborne station.

Aerialtriangulation See aerotriangulation.

Aerotriangulation The determination of horizontal and vertical coordinates of points on the ground, from measurements performed using overlapping aerial photographs and from already known coordinates of points on the ground.

Altitude

- The vertical angle between a horizontal plane and the line to the observed or defined object. In surveying, a positive altitude (measured upward from the horizon) is termed an angle of elevation, and a negative altitude (measured downward from the horizon) is termed an angle of depression.
- Altitude is sometimes used to apply to elevation above a datum, the altitude of an airplane above ground, or above sea level.

Angle The difference in direction between two convergent lines. It may be classed as horizontal, vertical, oblique, or spherical, according to whether it is measured in a horizontal, vertical, or inclined plane, or in a curved surface.

Angle, central

- The angle at the center of a circular arc that passes through beginning (PC) and end (PT) of the arc. This angle is also equal to the change in direction of the tangents to the arc that pass through the PC and PT. The central angle is equal to the deflection angle at the PI. In route surveys, it is commonly referred to as delta.
- For spirals the central angle is called theta.

Angle, deflection A horizontal angle measured from the prolongation of the preceding line, right or left, to the following line.

Annotated photography A photograph on which information has been added to identify, classify, outline, clarify, or describe features that would not otherwise be apparent in examination of an unmarked photograph.

Aperture

- An opening; particularly that opening in the front of a camera through which light rays pass when a picture is taken, or
- The diameter of the objective lens of a telescope or other optical instrument, usually expressed in inches, but sometimes as the angle between lines from the principal focus to opposite ends of a diameter of the objective lens.

Azimuth The horizontal angle measured clockwise between the reference meridian and the line to an observed or described point. See “Grid azimuth,” “Magnetic azimuth,” and “Back azimuth”.

Azimuth mark A mark set at a significant distance from a triangulation or traverse station to mark the end of a line for which the azimuth has been determined, and to serve as a starting or reference azimuth for later use.

Azimuth pair Two monuments between which the azimuth (bearing) is known. Generally used for obtaining the starting or ending direction of a traverse

B

Back azimuth The azimuth of a line at the end opposite the reference end. Back grid azimuths differ by 180 degrees from forward grid azimuth. Back geodetic azimuths differ from the forward geodetic azimuth by 180 degrees minus the convergence angle.

Backsight

- In traversing, a backsight (BS) is a sight to a previously established point.
- In leveling, a backsight is a reading to a point whose elevation has been previously determined.

Base line Also see "Line, base (USPLS) " A base line is:

- a surveyed line established with more than usual care, to which surveys are referred for coordination and correlation, or
- (for construction) the centerline of a railway or highway. A reference line for the construction of a bridge or other structure.
- a baseline consists of a pair of stations for which simultaneous GPS data has been collected.
- An NGS calibrated base line, which is used to detect random or systematic errors associated with a total station.
- **Base Station** Also called a reference station. A known location specifically to collect data for differentially correcting rover files or the relative processing of rover files.

Bearing The directions of one point or object with respect to another, where the direction of the line is expressed by the acute angle with respect to the reference meridian. Bearings are measured clockwise or counterclockwise from north or south, depending on the quadrant.

Bench mark A relatively permanent object, natural or artificial point whose elevation above or below an adopted datum is known. See "Temporary bench mark".

Bridging The extension and adjustment of a photogrammetric survey between areas with ground control.

C

C-factor An empirical evaluation, which expresses the vertical (elevation) measuring capability of a stereoscopic system; generally defined as the smallest contour interval that can be plotted to required accuracy. The C-factor is often used to determine the flight height from which aerial photographs should be taken for photogrammetrically accomplishing topographic mapping, at the smallest contour interval accurately plottable from using a particular aerial camera and instrument system. The practicable flight height is the contour interval multiplied by the C-factor.

Cairn An artificial mound of rocks, stones, or masonry, usually conical or pyramidal, whose purpose is to designate or to aid in identifying a point of surveying or cadastral importance.

Call (USPLS) A reference to, or statement of, an object, course, distance, or other matter of description in a survey or grant requiring or calling for a corresponding object, or other matter of description, on the land.

Cartography The art and science of expressing graphically, by maps and charts, the known physical features of the earth, or of another celestial body; usually includes the works of mankind and his varied activities

Centerline As applied to a street, or alley a line midway between the sidelines. As applied to a highway the line from which other elements such as right-of-way and edge of pavement is determined. See Present Traveled Way.

Chain A unit of length used in the subdivision of public lands of the United States. The Gunter’s chain is 66 feet long and is divided into 100 links, each link being 0.66 ft long (early chains were 50 links, or 33 feet in length).

Chaining Measuring distances on the ground using a chain or tape. Chaining and taping are generally considered synonyms.

Check structure (irrigation) A structure that can be used to raise the level of water in an irrigation canal for turnouts.

Chord A straight line connecting two points on a curve.

Circle (surveying instrument) The circular disc of a surveying instrument, which is perpendicular to and centered about an axis of rotation.

Circle position A prescribed setting (reading) of the horizontal circle of a total station, to be used for the observation on the initial station of a series of stations that are to be observed.

Circuit closure In leveling, the amount by which the algebraic sum of the measured differences of elevation fails to equal the theoretical closure, zero.

Clockwise angle A horizontal angle measured to the right. A clockwise angle may have any value between zero degrees and 360 degrees. Azimuths are clockwise angles generally measured from either north or south.

Closed traverse A traverse that begins at a known or adopted position and closes back to the same beginning point or closes to a second or subsequent known point. The preferred method is to begin at one known control point and close to a second or subsequent known control point. This gives a check of the orientation of the traverse.

Closing error See “Error of closure”.

Closing the horizon Measuring the last of a series of horizontal angles at a station. At any station, the sum of the individual horizontal angles between adjacent lines should equal 360 degrees. The amount by which the sum of the measured angles fails to equal 360 degrees is the angular error of closure. This error is distributed as a correction among the measured angles to make their sum exactly 360 degrees. The error and the correction have opposite algebraic signs.

Collimation, error of The angle between the line of collimation (line of sight) of a telescope and its collimation axis. When the collimation adjustment of an instrument is perfect (which is never the case), the line of collimation and the collimation axis will coincide, and the error of collimation will be zero. In practical work, the adjustment is carried to where the error is so small that it can be considered to be negligible. Error of collimation is a systematic error and in a series of observations is usually treated as being of the constant error type.

Combination Scale Factor (CSF) The sea level factor x grid scale factor. Used to determine ground distances from state plane grid distances.

Commencing, point of In metes and bounds description, the starting point if not a part of the tract being described, e g, “Commencing on the northwest corner of Section 12, T 44N, R 67W, 6th P M, thence southerly along the west line of said section 500 feet to the point of beginning”.

Compass adjustment A method used to distribute random errors in survey measurements such as a traverse. Based on the assumption that after the angular error is distributed the closure is a result of distance measurements only.

Contour interval The difference in elevation between adjacent contour lines on a map.

Contour line A line on a map representing equal elevation.

Contour map see map, contour.

Control A system of points whose relative positions have been determined from survey data. See “Basic control,” “Horizontal control,” and “Vertical control”.

Control, geodetic A system of horizontal and/or vertical control stations that have been established and adjusted by geodetic methods and in which the shape and size of the earth has been considered in position computations.

Control, horizontal Control marks (points) with horizontal positions only. The positions may be referenced to either geodetic or plane coordinates.

Control, level A series of bench marks or temporary bench marks of known elevation, established throughout a project.

Control, photo Any station in a horizontal and vertical control system that is identified on a photograph and used for correlating the data shown on that photograph; also termed photo control point, picture control point, and ground control point.

Control point Also called a control mark. A monumented point to which coordinates have been, or are being assigned by the use of surveying observations. The National Geodetic Survey maintains a nation-wide set of control points. A station whose position (horizontal or vertical) has been determined from survey data, and is used as a base for a dependent survey.

Control survey A survey that provides positions (horizontal or vertical) of points to which supplementary surveys are adjusted.

Control traverse A main scheme closed traverse, which is performed to the highest degree of precision used on a project. The control traverse is used as the horizontal datum base for all aspects of a project. Secondary traverses to such things as topographic features, aerial targets and property corners are tied to the control traverse.

Conventional Survey

- A survey using instruments such as total stations and digital levels.
- A centerline survey where data is recorded by stations and offsets. This type generally is restricted to overlay projects.

Coordinates Linear or angular quantities, or both, which designate the position of a point in relation to a given reference frame. There are two general divisions of coordinates used in surveying: polar coordinates and rectangular coordinates. These may be subdivided into three classes: plane coordinates, grid coordinates, and geodetic coordinates.

Corner A point on a land boundary, at which two or more boundary lines meet. Can be either a property corner or a property controlling corner, or a public land survey corner or any combination of these. Not the same as monument, which refers to the physical evidence of the corner's location on the ground.

Corner (USPLS) A point on the surface of the earth, determined by the surveying process, marking an extremity of a boundary of a subdivision of the public lands, usually at the intersection of two or more surveyed lines; often incorrectly used to denote the physical structure, or monument, erected to mark the corner point. Corners are described in terms of the points they represent.

Corner, closing (USPLS) A corner at the intersection of a surveyed line with a previously established boundary line. To compensate for convergence of the meridians, standard parallels (once termed "correction lines") are established at intervals of 24 miles. These were formerly established at varying intervals up to 60 miles. Corners established at the time the standard parallel is run are termed "standard corners" and generally govern the surveys to the north. A second set of corners — closing corners — is established at the point where the subsequent survey lines intersect a previously establish line. Closing corners are also established at the boundaries of reservations, grants, claims, and other previously surveyed or segregated tracts of land.

Corner, double (USPLS) Normally the two sets of corners along a standard parallel; the standard township, sections, and quarter-section corners placed at regular intervals of measurement; additionally, the closing corners established on the line at the points of intersection of the guide meridians, range and section lines of the surveys brought in from the south.

Corner, existent (USPLS) A corner whose position can be identified by verifying the evidence of the monument, or its accessories, by reference to the description that is contained in the field notes, or where the point can be located by an acceptable supplemental survey record, some physical evidence, or testimony. Even though its physical evidence may have entirely disappeared, a corner will not be regarded as lost if its position can be recovered through the testimony of one or more witnesses who have a dependable knowledge of the original location.

Corner, found A term adopted to designate an existent monument that has been recovered by field investigation.

Corner, lost (USPLS) A previously established survey corner whose positions cannot be recovered beyond reasonable doubt, either from traces of the original marks or from acceptable evidence or testimony that bears on the original position, and whose location can be restored only by reference to one or more interdependent corners.

Corner, property A geographic point on the surface of the earth and is on, a part of, and controls a property line.

Corner, property controlling (USPLS) A public land survey corner or any property corner which does not lie on a property line of the property in question but which control the location of one or more of the property corner of the property in question.

Corner, meander (USPLS) A corner established at the intersection of standard, township or section lines with the banks of navigable streams or any meanderable body of water. See also “Corner, (USPLS)”.

Corner, obliterated (USPLS) A corner at which there are no remaining traces of the monument or its accessories, but whose location has been perpetuated or may be recovered beyond reasonable doubt, by the acts and testimony of the interested landowners, competent surveyors, other qualified local authorities, witnesses, or by some acceptable record evidence. A position that depends upon the use of collateral evidence can be accepted only as duly supported, generally through proper relation of known corners, and by agreement with the field notes regarding distances to natural objects, stream crossings, line trees, and off-line tree blazes, etc, or unquestionable testimony.

Corner, witness (USPLS) A witness corner, by conventional usage, is a monumented point usually on a line of the survey and near a corner. It is employed in situations where it is impracticable to occupy the site of a corner. When the true point for a corner falls upon an inaccessible place (such as within an unmeandered stream, lake, or pond, or in a marsh, or upon a precipitous slope of cliff), where the corner cannot be occupied, a witness corner will be established at some suitable point where the monument may be permanently constructed, but preferably on a line of the survey. Usually only one witness corner will be established in each instance and it will be located upon any one of the surveyed lines leading to a corner, if a secure place within a distance of 10 chains is available. If there is not a place to be found on a surveyed line within that limiting distance, that can be occupied and marked, a witness corner may be located in any direction within a distance of 5 chains. If a monument replacement is involved using witness corner, and a specific distance given is a direct tie, no form of proration is acceptable.

Crab The condition, which occurs when the sides of an aerial photograph are not parallel or perpendicular to the direction of flight. It results when the aircraft's longitudinal axis does not coincide with the flight direction.

Curve, circular A curve of constant radius. All points on the curve are equal distance from the center of the circle.

Curve, compound Name for two circular curves of different radii, which are tangent at one point with both curves lying on the same side of the common tangent.

Curve, degree of The degree of curve (D) defines the radius of a highway or railroad circular curve. There are two definitions:

- (chord) The angle subtended at the center of a circle by a chord of 100 feet, and
- (arc) The angle subtended at the center of a circle by an arc of 100 feet.

Definition 1 was used in railroad and early highway design. Definition 2 is used in present-day engineering of highway design

D

Datum Any numerical or geometrical quantity or set of such quantities that may serve as a reference or base for other quantities.

Datum, horizontal The position on the spheroid of reference assigned to the horizontal control (triangulation and traverse) of an area and defined by:

- the position (latitude and longitude) of one selected station in the area, and
- the azimuth from the selected station on an adjoining station.

The horizontal-control datum may extend over a continent or be limited to a small area. A datum for a small area is usually called a local datum. The horizontal-control datum for the North American continent is known as the North American Datum of 1983 (NAD83).

Datum, mean sea level A determination of mean sea level that has been adopted as a standard datum for heights or elevations. The NGVD29, is an example. Mean sea level datum was based on tidal observations over a number of years at various tide stations along the coasts. NGVD29 has been superceded by NAVD88.

Datum, state plane coordinates The surface onto which each point is projected mathematically from the corresponding point on the spheroid to its corresponding point on a flat surface. For illustrative purposes, the Lambert conformal projection is represented by a cone and the Transverse Mercator projection by a cylinder.

Deflection angle A horizontal angle measured from the prolongation of the preceding line, clockwise or counterclockwise as necessary, to the following line.

Departure In a plane survey, the amount that one end of a line is east or west of the other end. Also equal to the difference between the X coordinate of the two ends of the line. It is equal to the length of the line times the sine of the bearing.

Digital Terrain Model (DTM) An electronic three-dimensional map of a land surface.

Direction The angle between a line and an arbitrarily chosen reference line. When the reference line is north or south and the angle is designated east or west and less than 90 degrees, the direction is called the bearing. When the reference line is north or south and the angle is clockwise between 0 degrees and 360 degrees, the direction is called the azimuth.

Directional instrument A theodolite or total station in which the horizontal circle remains fixed during a set of observations and the direction of each point of the instrument is read on the circle. Horizontal angles cannot be repeated or accumulated on a direction instrument, but the circle can be advanced in position between successive sets of observations.

Direct reading The reading of the horizontal or vertical circle of a theodolite or total station the instrument direct orientation. In field notes, the letter “D” preceding the observed value indicates a direct reading.

Discrepancy

- The difference between duplicate or comparable measures of a quantity, or
- the difference between computed values of a quantity obtained by different processes in the same survey.

Displacement, height Displacement of images radially inward or outward with respect to the photograph nadir, depending on whether the ground objects are, respectively, below or above the elevation of the ground nadir.

Displacement, relief Displacement of images radially inward or outward from the nadir point of the photograph. Relief displacement of images is caused by differences in elevation of the corresponding ground objects whether below or above, respectively, the elevation of the ground nadir.

Displacement, tilt Displacement of images on a tilted photograph radial from its isocenter. Tilt displacement is outward or inward with respect to the isocenter, according to whether the images are on the low or high side of the isometric parallel (the low side is the closest to the earth, or the object plane).

Distortion Any shift in the position of an image on a photograph that alters the perspective characteristics of the photograph. Causes of image distortion include lens aberration, differential shrinkage of film or paper, and motion of the film or camera.

Diversion box (irrigation) Usually a concrete structure that allows irrigation water to be routed in several different directions.

Diversion structure (irrigation) Similar to a turnout, but used to divert water from a river or stream into the beginning of an irrigation canal.

Double centering A method of prolonging a line from a fixed point whereby the backsight is taken with the telescope in the direct position. The telescope is placed in the inverted position and the foresight is made. The point at which the vertical cross hair intersects the hub is then marked. The instrument is then rotated 180 degrees to take a backsight with the telescope in the inverted position, and a second projected point with the telescope in the direct position is marked on the hub. A point midway between the two marked points is the true point on the prolonged line.

Double proportionate measurement A method for restoring a lost corner of four townships or a lost interior corner of four sections. It is based on the principle that monuments north and south should control the latitudinal position of a lost corner, and monuments east and west should control the longitudinal position. In this method, the influence of one identified original corner is balanced by the control of a corresponding original corner upon the opposite side of a particular missing corner, which is to be restored, each identified original corner being given a controlling weight inversely proportional to its distance from the lost corner.

Drift The lateral shift or displacement of the aircraft from the planned flight path due to the action of the wind, navigational errors or other causes.

Drop structure (irrigation) Usually concrete, steps in an irrigation canal used to flatten canal slope between steps.

E

Easting (East) A linear distance eastward from the north-south line that passes through the origin (or false origin) of a grid. Generally, the false origin is selected so that all east coordinates are positive. Also referred to as the X-coordinate.

Elevation The vertical distance from a datum, such as NAVD88 to a point or object on the earth's surface.

Emulsion (photography) A suspension of a light-sensitive silver salt (especially silver chloride or silver bromide) in a colloidal medium (usually gelatin), which is used for coating photographic films, plates, and papers. Types of photographic emulsions currently in common usage are panchromatic (black and white), color negative, color positive, infrared color, and infrared black and white.

Endlap The overlap area common to two successive aerial photographs in a strip.

Ephemeris

- A publication giving coordinates of celestial bodies at uniform time intervals; the coordinates are usually given for one calendar year. A publication giving similar information in a form suitable for use by a navigator is called an almanac.
- A data file that contains orbit information on all satellites, clock corrections, and atmospheric delay parameters. It is transmitted by a GPS satellite to a GPS receiver, where it facilitates rapid satellite vehicle acquisition within GPS receivers. Almanac data must be acquired before GPS navigation can begin.

Error of closure The amount by which the value of a quantity obtained by surveying operations fails to agree with another value of the same quantity held fixed from earlier determinations or with a theoretical value of the quantity.

Extrapolation The process of estimating the value of a quantity beyond the limits of known values by assuming that the rate of change between the last few known values continues.

Eye Base The interpupillary distance between the eyes of an individual.

F

Fiducial marks (photogrammetry) Index marks, at least four, rigidly connected with the camera lens through the camera body and forming images on the negative that generally define the principal point of the photograph. Also, those marks, usually four in number, in any instrument that define the axes whose intersections fix the principal point of a photograph or negative and fulfill the requirements of interior orientation.

Flight height The height of the camera above the mean elevation of the ground at the instant of exposure.

Flight line A line drawn on a map or chart to represent the track of the aircraft during the period of taking aerial photographs.

Flight map A map showing the lines along which an aircraft is to fly when aerial photographs are being taken and/or the locations at where photographs are to be taken.

Flight strip A succession of overlapping aerial photographs taken along a single course.

Focal length A general term for the distance between the center, vertex, or rear node of a lens (or the vertex of a mirror) and the point at which the image of an infinitely distant object comes into critical focus. The term must be preceded by an adjective such as “equivalent” or “calibrated” to have precise meaning.

Foresight

- A point to which an instrument sighting is made for measuring or establishing its elevation and/or its horizontal positions.
- In leveling, the foresight is an observation made to a point of unknown elevation.

G

Gap (aerial photography) Any space between aerial photographs failing to meet minimum ground coverage requirements. The gap may be space not covered by any photograph in line of flight or between separate flights, or the space where specified endlap or sidelap was not obtained.

General Land Office Formerly an office of the United States government, being a division of the Department of the Interior, having charge of all executive action relating to the public lands, including their survey, sale or other disposition, and patenting; originally constituted by Act of Congress in 1812. The General Land Office and the US Grazing Service were consolidated into the Bureau of Land

Management under the Department of the Interior by the 1946 Reorganization Plan No 3, 403.

Geodetic surveys Surveys in which the shape of the earth is considered.

Geoid Theoretical continuous surface that is perpendicular at every point to the direction of gravity.

Global Positioning System (GPS) A system for providing precise location, which is based on data transmitted from a constellation of satellites.

Grid A surface that consists of imaginary parallel lines that intersect at right angles. The term is frequently used to designate a plane-rectangular coordinate system.

Grid azimuth An azimuth measured from grid north.

Grid coordinates The X (E) and Y (N) values that designates a point on a grid.

Grid factor A scale factor used to convert ellipsoid distances to grid distance and vice versa.

Grid length The distance between two points as obtained by computation from the plane-rectangular coordinates of the points. In the state coordinate systems, a grid length differs from a geodetic distance by the amount of a correction based on a grid scale and elevation scale.

Grid tick A small mark placed at the edge of a map or drawing to indicate a measurement. The grid system used may be indicated by ticks for future reference.

Ground control In photogrammetry, control obtained from surveys as distinguished from control obtained by photogrammetric methods.

Guard stake A stake driven near (within 6 inches) a hub, usually sloped with the top driven to clear the hub. The guard stake protects the hub and the markings on the stake identify it.

H

Head gate (irrigation) Generally a concrete structure that includes slide gates, screw gates or turnouts.

Height of instrument

- The height of the center of the telescope (horizontal axis) above the ground or station mark.
- (Leveling) The height of the line of sight of the leveling instrument above the adopted datum.

Height of signal Vertical distance from the top of the control point to the center of the object observed, for example the distance from the top of the control point to the center of a prism.

Highwater mark The visible mark left on a pier, abutment, rock, etc. indicating the water surface level (elevation) during a flood.

Horizontal angle An angle measured in a horizontal plane.

Horizontal axis (total station) The axis about which the telescope of a total station rotates when moved vertically. It is the axis of rotation that is perpendicular to the vertical axis of the instrument and to the axis of the collimation of the telescope. It should coincide with the line through the centers of the pivots that support the telescope. For an instrument in complete adjustment, it occupies a horizontal position, and when the telescope is rotated around it, the axis of collimation will define the vertical plane; deviations from this condition are measured with a striding level or with a hanging level.

Horizontal control Control stations whose horizontal coordinates are known.

Horizontal datum In plane surveying, the grid system of reference used for the horizontal control of an area; defined by the easting and northing of one station in the area, and the azimuth from this selected station to an adjacent station.

Horizontal direction A direction in a horizontal plane.

Horizontal distance The distance measured in a horizontal plane, as distinguished from a distance measured on a slope.

Horizontal plane A plane perpendicular to the direction of gravity.

Horizontal position The grid position of a horizontal control point.

Horizontal and vertical control point When used in aerial photo work, it refers to a photo image point that has X, Y and Z coordinate values. These image points are used to control the scale and elevation of stereo models in the preparation of topographic maps.

Hub A wooden stake with a tack or other marker to indicate the exact position. A guard stake protects and identifies the hub.

I

Intersection A method of locating the horizontal position of a point by observations from two or more points of known position. A point whose horizontal position is located by intersection is known as an intersection station.

Instrument direct The theodolite or total station when oriented in its normal position. In field notes, "D" signifies a reading with the instrument direct, called a "direct reading".

Instrument reversed or inverted The theodolite or total station when the telescope is inverted from it normal position. In field notes, "R" signifies a reading with the instrument reversed, called a "reversed reading".

Inverse computation The computation for the length and azimuth of a line from the coordinates of its endpoints. This is the inverse of the survey computation in which the position of a new station is determined through the length and azimuth of the connecting line.

L

Latitude

- Angular distance measured on a meridian, north or south from the equator.
- In plane surveying, the amount that one end of a line is north or south of the other end. Also equal to the difference between the Y coordinate of the two ends of the line.
- It is equal to the length of the line times the cosine of the bearing.

Least squares An adjustment method to distribute random errors associated with conventional survey or a GPS network. The adjustment minimizes the sums of the squares of the residual. Generally preferred over the compass adjustment.

Level

- Pertaining to a level surface
- To make horizontal at the point of observation
- An instrument for leveling

Level loop The measurement of elevations by differential leveling from a known bench mark to another known bench mark. A level loop may also commence at a known bench mark and close on the same bench mark.

Level datum A level surface to which elevations are referred. The generally adopted level datum for leveling in the United States is NAVD88. For local surveys an arbitrary level datum is often adopted and defined in terms of an assumed elevation for some physical mark (bench mark).

Level line

- A line in a horizontal plane, or
- A line over which leveling operations are accomplished.

Level net Lines of spirit leveling connected together to form a system of loops or circuits extending over an area. Also called a vertical control net.

Level surface A horizontal plane.

Leveling The operation of measuring vertical distances, directly or indirectly, to determine elevations.

Leveling, differential The process of measuring the difference of elevation between any two points by spirit leveling.

Leveling, reciprocal A procedure for measuring zenith angles and distances from both ends of a line for the determination of differences in elevations. This method compensates for earth curvature and the effects of atmospheric refraction.

Leveling, spirit The determination of elevations by use of a leveling rod and an instrument using a spirit level to establish a horizontal line of sight; the term has now been broadened to include leveling by means of other types of levels, such as a pendulum level and digital levels.

Leveling, trigonometric The determination of differences in elevation trigonometrically from observed zenith angles and measured slope distances.

Leveling rod A rod or bar with a flat face graduated in linear units with zero at the bottom, used in measuring the vertical distance between a point on the ground and the horizontal line of sight of a leveling instrument.

Line, base (USPLS) A line extending east and west along the astronomic parallel passing through the initial point, along which standard township, section, and quarter-section corners are established. As may be inferred from its designation, the base line is the line from which is initiated the survey of the township boundaries and section lines. Auxiliary governing lines, known as standard parallels or correction lines, are established along astronomic parallels usually at intervals of 24 miles north or south of the base line. In some of the early surveys, the base line was referred to as the "basis parallel".

Line, grade The profile of the road, usually expressed as a percentage, which is the number of units of change in elevation per 100 units of horizontal distance.

Line, random A trial line, directed as closely as possible toward a fixed terminal point that is not visible from the initial point. The error of closure (amount by which the second station is missed) permits the computation of a correction to the initial azimuth of the random line; it also permits the computation of offsets from the random line to establish points on the line between the survey stations.

Longitude The angle between the plane of a given meridian and the plane of an arbitrary initial meridian, generally the meridian of Greenwich. It may be measured as the angle at the poles between the two meridians, as the arc of the equator intercepted between the meridians, or as the arc of a parallel of latitude intercepted between the meridians.

Loop traverse A closed traverse that starts and ends at the same station.

M

Magazine (photography) A container for rolled film or photographic plates attached to the camera body and usually equipped with automatic mechanisms to advance and position the photographic material for exposure.

Magnetic azimuth An azimuth measured with reference to the direction indicated by a magnetic compass needle. Magnetic azimuth is measured from magnetic north, which is east or west of true north as shown by the magnetic declination.

Magnetic declination The angular amount that a magnetic compass needle points eastward or westward from north.

Magnetic variation Regular or irregular changes in the magnetic declination with time.

Map A representation on a plane surface, at an established scale, of the physical features (natural, artificial, or both) of a part or the whole of the earth's surface, by the use of signs and symbols, and with the method of orientation indicated. Also, a similar representation of the heavenly bodies. A map may emphasize, generalize, or omit the representation of certain features to satisfy specific requirements. The type of information, which a map is designed primarily to convey, is frequently used, in adjective form, to distinguish it from maps of other types. A map should contain a record of the projection upon which it is constructed.

Map, base

- A map showing certain fundamental information, copies of which are used to compile additional data of specialized nature. Often used to define a large-scale planimetric map compiled from aerial photographs, copies of which are used for the addition of contours and other data by use of the plane table and photogrammetric methods.
- A map containing all the information from which maps showing specialized information can be prepared; a master map.

Map, cadastral A map showing the boundaries of subdivisions of land, usually with the bearings and lengths thereof and the areas of individual tracts, for purposes of describing and recording ownership. A cadastral map may also show culture, drainage, and other features relating to the value and use of land.

Map, contour A map that portrays the elevation and configurations of the ground by contour lines, and lacks any other details except notations and contour elevations.

Map, engineering A map showing information that is essential for planning an engineering project or development and for estimating its cost. An engineering map is usually a large-scale map of a comparatively small area or of a route. It may be entirely the product of an engineering survey, or reliable information may be collected from various sources for the purpose and assembled on a base map.

Map, highway A map containing the planimetric configurations of a highway and its connections, at grade or by controlled access, to other highways; also a map containing the details of curvature, roadside and cross drainage, right-of-way boundaries, and delineations regarding adjacent occupancy and land uses. The detail in a highway map is dependent entirely upon its scale and the purpose of its use whether for merely indicating travel routes or for depicting construction details.

Map, planimetric A map that presents only the horizontal positions for the features represented; distinguished from a topographic map by the omission of relief in measurable form.

Map, projection, Lambert conformal conic A conformal map projection of the so called conical type, on which all geographic meridians are represented by straight lines which meet in a common point outside the limits of the map, and the geographic parallels are represented by a series of arcs of circles having this common point for a center. Meridians and parallels intersect at right angles, and angles on the earth are correctly represented on the projection. This projection may have one standard parallel along which the scale is held exact; or there may be two such standard parallels, both maintaining exact scale. At any point on the map, the scale is the same in every direction. It changes along the meridians and is constant along each parallel. Where there are two standard parallels, the scale between those parallels is to small, beyond them to large.

Map scale The relationship existing between a distance on a map and the corresponding distance on the earth. Map scale may be expressed as an equivalence, usually by different units, that is, 1 inch = 1 mile, as a numerical fraction or ratio (1/63,360 or 1:63,360), or graphically, by a bar scale. Fractional map scale is representative in any linear units, and is often called the representative fraction, or R F, when the numerator is unity.

Map scale, graphic (or bar) A line on a map subdivided and marked with the distance which each of its parts represents on the earth.

Map, topographic A map that presents the horizontal and vertical positions of the features represented; distinguished from a planimetric map by the addition of relief in measurable form. A topographic map usually shows the same features as a planimetric map but uses contours or comparable symbols to show mountains, valleys, and plains; and in the case of hydrographic charts, symbols and numbers to show depths in bodies of water.

Mark (Point) A definite object (such as an imprinted metal disk) used to designate a survey mark or point. Usually used with a qualifying term such as station mark, reference mark, azimuth mark, or bench mark. Sometimes refers to the entire survey monument.

Mass diagram The earthwork mass diagram is a continuous graph of net cumulative yardage at any point on an earthwork project. It is used to analyze amounts of excavation and embankment, balance points, and haul requirements.

Mean sea level (MSL) The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings. A determination of mean sea level that has been adopted as a standard for heights is called a sea level datum. It was based upon observations taken over a number of years at various tide stations along the coasts of the United States and Canada.

Memorial (USPLS) A durable article deposited in the ground at the position of a corner to perpetuate that position should the monument be removed or destroyed. The memorial is usually deposited at the base of the monument and may consist of anything durable, such as glass or stoneware, a marked stone, charred stake, or a quantity of charcoal.

Meridian A north-south line from which longitudes (or departures) and azimuths are reckoned.

Meridian, central

- The line of longitude at the center of a projection. One of the parameters for construction of a projection.
- (State plane coordinate system) The meridian used as the y-axis for computing projection for a state coordinate system. The central meridian of the system usually passes close to the center of projection.

Metes and bounds A method of describing land by measure of length (metes) or the boundary lines (bounds). The most common method is to recite direction and length of each line as one would walk around the perimeter. In general, the “metes” and “bounds” can be recited by reference to record, natural or artificial monuments at the corners; and record, natural or cultural boundary lines. See also “Description, metes and bounds”.

Monument (USPLS) A physical structure that marks the location of either a property corner or a property controlling corner or other survey points. In public-land surveys, the term “corner” is employed to denote a point determined by the surveying process, whereas the “monument” is the physical structure erected to mark the corner point upon the earth’s surface. Monument and corner are not synonymous, though the two terms are often used in the same sense.

Monument, record An adjoiner property called for in a deed such as a street or particular parcel of land. Frequently the boundary line of the adjoiner is referred to as the record monument; actually the entire property, rather than the line, is the monument. Physical monuments may or may not mark a record monument.

Mosaic, photographic An assembly of aerial photographs whose edges usually have been cut and matched to form a continuous photographic representation of a portion of the earth's surface. Often called an aerial mosaic. "Controlled mosaic" — a mosaic oriented and scaled to horizontal ground control; usually assembled from rectified photographs. "Uncontrolled mosaic" — a mosaic in which the photographs have not been adjusted by the reference to ground control.

Most probable value That value of a quantity that is mathematically determined from a series of observations and is more nearly free from the effects of blunders and errors than any other value that might be derived from the same series of observations.

Multipath Interference caused by reflected GPS signals arriving at the receiver, typically as a result of nearby structures or other reflective surfaces.

N

Nadir The point on the celestial sphere directly beneath the observer, and directly opposite the zenith.

Neat model The common area between two aerial photographs lying between the principal point of each photograph.

Normal In general, a straight line perpendicular to a surface or to another line. Also, a condition of being perpendicular to a surface or line. In geodesy, a straight line perpendicular to the surface of the spheroid.

North The primary reference direction relative to the earth; the direction indicated by 0 degrees in any system. A term used to define:

- an astronomic meridian,
- a geodetic meridian,
- the direction of north from magnetic north corrected for declination,
- the cardinal directions run in the Public Land Survey

The use of the term "True North" should be avoided.

North arrow An arrow-like symbol indicating the direction and the type of meridian to which the control framework of a map or drawing is referenced. Auxiliary arrows may be shown indicating the direction of other meridians that may be of interest to the user of the map.

North, magnetic The direction of the north-seeking end of a magnetic compass needle not subject to transient or local disturbance.

Northing (North) A linear distance northward from the east-west line that passes through the origin (or false origin) of a grid. Generally the false origin is selected so that all northings remain positive. Same as y-coordinate.

O

Objective The lens in a microscope or telescope that is nearest the object. Also the lens used in a camera.

Oblique photograph (aerial) An aerial photograph taken with the camera axis intentionally tilted between three and ninety degrees from vertical.

Observed value A value of a quantity that is obtained by instrumental measurement of the quantity. The term “measured” is often applied to the value of a quantity derived from instrumental measurement after corrections have been applied for systematic errors.

Occupied station A traverse or some other mark over which a total station or GPS antenna is set up for the measurement and recording of survey data.

Offset line A supplementary line close to, and usually parallel to, a main survey line to which it is referenced by measured offsets. When the line for which data are desired is in such position that it is difficult to measure over it, the required data are obtained by running an offset line in a convenient location and measuring offset from it to salient points on the other line.

Open traverse A traverse that starts at a point of known or assumed position, and ends at a point whose relative position is unknown with respect to the starting point.

Order of accuracy A mathematical ratio defining the general accuracy of the measurements made in a survey.

Orthophoto A photo reproduction that has been corrected for camera lens distortions, tilt, and topographic displacement. Because of these corrections, objects in an orthophoto appear in their true orthographic position.

Overlap

- The amount by which one photograph includes the same area as covered by another, customarily expressed as a percentage.
- That area of a map or chart, which overlaps the same geographical area on an adjoining map or chart.

- An area included within two surveys of record, which by the record is described as having one or more common boundary lines with no inclusion of identical parts.

Overlapping pair (photogrammetry) Two photographs taken at different exposure stations in such a manner that a part of one photograph shows the same terrain as shown on a part of the other photograph. This term covers the general case and does not imply that the photographs were taken for stereoscopic examination.

Overlay (mapping) A record on a transparent medium to be superimposed on another record. For example, maps showing original land grants (or patents) prepared as tracing-cloth overlays in order that they can be correlated with the maps showing present ownership. Also, any of the several overlays that may be prepared in compiling a manuscript map, usually described by name, for example, lettering overlay

P

Parallax The apparent displacement of the position of any point with respect to a reference point or system, caused by a shift in the station of observation. The parallax of one point in space in respect to a reference point is the angle of convergence of the rays from two observation stations to the reference point, minus the angle of convergence of the rays from the same two observation stations to the second point. On a pair of photographs of the points in space taken from the two observation stations, parallax is measured by distances on the photographs rather than by angles.

Parallax, instrumental A change in the apparent position of an object with respect to the reference marks of an instrument which is due to imperfect adjustment of the instrument, to a change in the position of the observer, or both.

Parallel, geographic A line on the earth having the same latitude.

Parallel, standard (USPLS) An auxiliary governing line initiated at a selected township corner on a principal meridian, usually at intervals of 24 miles from the base line, on which standard township, section, and quarter-section corners are established.

Pass point A natural image or marked point visible on a photograph for which the horizontal position and/or elevation has been or will be determined by photogrammetric use of the photograph and its stereoscopically adjacent photographs. A pass point is used for the orientation of photographs in photogrammetric instruments in the same manner in which supplemental control points are used.

Perpendicular A perpendicular line, plane, etc. A distinction is sometimes made between perpendicular and normal, the former applying to a line at right angles to a straight line or place, and the latter referring to a line at right angles to a curve or curved surface.

Phase The visible aspect of an object. The apparent displacement of an object or signal caused by one side being more strongly illuminated than the other. The resultant error in pointing is similar to the error caused by observing an eccentric signal.

Phase of target (error caused by) Uneven illumination of target causing error in sighting.

Photo I D point The photo identification point is a photo image point that is visible on the photograph and identifiable on the ground. Photo ID points can be pre-marked (targeted) points, but more commonly they are definable physical features. The term is generally used to refer to photo image points that are used to supplement control after photos have been taken.

Photogrammetry The art, science, and technology of obtaining reliable information about physical objects and the environment through the processes of recording, measuring, and interpreting photographic images and patterns of electromagnetic radiant energy and other phenomena.

Photograph A general term applying to either a positive or a negative exposed on light sensitized material by use of a camera. Also the print made photographically from the negative or positive. The photograph may be exposed or printed, using any one of these types of emulsions: panchromatic, negative or positive color, infrared color, or infrared black and white.

Photographic interpretation The determination of the nature and description of objects that are imaged on a photograph.

Pier An intermediate support for the adjacent ends of two bridge spans.

Plane coordinates See “Grid coordinates”.

Plane survey A survey in which the effect of the curvature of the earth is neglected, and computations of the relative positions of the stations are made using the principles of plane geometry and plane trigonometry.

Plat (USPLS) The term "plat," as employed by the US Bureau of Land Management, refers to the drawing that represents the particular area included in a survey, such as a township, private land claim, or mineral claim, and the lines surveyed, established, retraced, or resurveyed, showing the direction and length of each line; the relation to the adjoining official surveys; the boundaries, descriptions, and area of each parcel of land subdivided; and, as nearly as may be practicable, a representation of the relief, and improvements within the limits of the survey.

Plat, supplemental (USPLS) A plat prepared entirely from office records designed to show a revised subdivision of one or more sections without change in the section boundaries and without other modification of the subsisting record. Supplemental plats are required where the subsisting plat fails to provide units suitable for administration or disposal, or where a modification of its showing is necessary. They are also required to show the segregation of alienated lands from public lands, where the former are included in irregular surveys of patented mineral or other private claims made subsequent to the plat of the subsisting survey, or where the segregation of the claims was overlooked at the time of its approval.

Plumb bob A conical device, usually of brass and suspended by a cord, by means of which a point can be projected vertically into space over relatively short distances.

Point of curve (PC) The point of change from a line to a curve.

Point of intersection (PI) The point of intersection of two lines.

Point of tangent (PT) The point of change from a curve to a line.

Point of vertical curve (PVC) The point of change from a line of uniform slope to a vertical curve.

Point of vertical intersection (PVI) The point of intersection of two lines, each having different uniform slope.

Point of vertical tangent (PVT) The point of change from a vertical curve to a line of uniform slope.

Point-transfer device A stereoscopic instrument used to mark corresponding image points on overlapping photographs.

Polaris The second-magnitude star, Alpha, in the constellation Ursa Minor (Little Dipper). Also known as the polestar, or North Star, because of its proximity to the north pole of the celestial sphere. Polaris is well situated for determinations of astronomical azimuth, and for the determination of the direction of the celestial meridian. It is at the extreme outer end of the handle of the "Little Dipper". See also "Pointers".

Position

- Data that define the location of a point with respect to a reference system.
- The place occupied by a point on the surface of the earth.
- The coordinates that define the location of a point on the geoid or spheroid.
- A prescribed setting (reading) of the horizontal circle of a total station that is to be used for the observation on the initial station of a series of stations to be observed.

Position, astronomical

- A point on the earth whose coordinates have been determined as a result of observations of celestial bodies.
- A point on the earth, defined in terms of astronomical latitude and longitude.

Position, geodetic A position of a point on the surface of the earth expressed in terms of geodetic latitude and geodetic longitude. A geodetic position implies an adopted geodetic datum. In a complete record of a geodetic position, the datum must be stated.

Positive (photography) A photograph having approximately the same rendition of light and shade as the original subject. A print from a negative.

Precision The degree of refinement in the performance of an operation, or the degree of perfection in the instruments and methods used when making the measurements. A measure of the uniformity or reproducibility of the result. Precision relates to the quality of the operation by which a result is obtained, and is distinguished from accuracy, which relates to the quality of the result.

Present Traveled Way The center of pavement. Not the same as centerline though often used interchangeably.

Pressure plate (photography) A flat plate, usually of metal but frequently of glass or other substance, which by the use of mechanical force, presses the film into contact with the focal plane of the camera.

Prima facie evidence Evidence deemed by law to be sufficient to establish a fact if the evidence is not disputed.

Print (photography) A photographic copy made by projection or contact printing from a photographic negative or from a transparent drawing, as in blueprinting. "Contact print" — a print made with the negative or transparent drawing in contact with the sensitized surface. "Ratio print" — a print, the scale of which has been changed from that of the negative by enlargement or reduction.

Profile A vertical section of the surface of the ground, or of underlying strata, or both, along any fixed line.

Profile, ground A line indicating ground elevations of a vertical section along a survey line.

Profile grade The trace of a vertical plane intersecting the top surface of the proposed wearing surface, usually along the longitudinal centerline of the roadbed. "Profile grade" means either elevation or gradient of such trace according to the context.

Progress sketch A map or sketch showing work accomplished. In triangulation and traverse surveys, each point that is established is shown on the progress sketch as well as lines observed over and base lines measured. In a leveling survey, the progress sketch shows the route followed and the towns passed through, but not necessarily the locations of the bench marks.

Prolongation With reference to a line it is used to indicate the continuity in the same direction of the recited course that is to be lengthened. A line is prolonged, but a curve is continued. A "prolongation" of a curve is the extension of the tangent to the curve, and should not be used in referring to curve continuation. The prolongation of a line, composed of several parts of different directional values, is the prolongation of the course nearest to the recited intersection or monument.

Property controlling corner Is a public land survey corner or any property corner which does not lie on a property line of the property in question but which controls the location of one or more of the property corners of the property in question.

Property corner A geographic point on the surface of the earth and is on, a part of, and controls a property line.

Proportionate measurement A measurement that applies an even distribution of a determined excess or deficiency of measurement, ascertained by retracement of an established line, to provide concordant relations between all parts.

Proportioning excess or deficiency (principle of) A principle, governed by several rules, of distributing excess or deficiency. For example, the frontage of 10 lots in a city block may total 1,000 feet by deed or plan. The measured length of the block is 1,007.42 feet. The excess, 7.42 feet, must be distributed among the 10 lots.

Prorate To divide or distribute proportionally.

Proration A method of distributing discovered excess of deficiency between parties having equal rights or proportionate rights to the excess or deficiency. A method of calibrating the tape of a recent surveyor against that of the original surveyor.

Q

Quarter line In public land surveys made on a township, range, and section basis, a quarter line is one of two lines joining opposite quarter section corners, and by the two lines the original survey of a section of land is divided into four parts. Also known as a mid section line.

R

Radial survey The measurement of angles and distances from a principal control point to a series of additional points, with the position of each point determined independent of the others.

Range (USPLS) Any series of contiguous townships situated north and south of each other; also sections similarly situated within a township. Ranges of townships are numbered consecutively east and west from a principal meridian; thus “range 3 east” indicates the third range or row of townships to the east from a principal meridian. The word “range” is used in conjunction with the appropriate township to indicate the coordinates of a particular township with reference to the initial point; thus “township 14 north, range 3 east” indicates the particular township which is the 14th township north of the base line and the third township east of the principal meridian.

Real Time Kinematic (RTK) The Differential GPS procedure whereby carrier phase corrections are transmitted in real time from a reference station to the user's roving receiver.

Reciprocal zenith angles Zenith angles observations taken with the instrument occupying both ends of a line for trigonometric leveling purposes in order to correct for the effects of refraction, parallax and earth curvature.

Reconnaissance A general examination or survey of a region with reference to its main features, usually as a preliminary to a more detailed survey.

Recording a deed The recording of deeds to give constructive notice of conveyance to purchasers and creditors. A deed may be valid between the grantor and grantee but will fail to give constructive notice to others if not so recorded.

Recovery of station To visit a survey station, identify its mark as authentic and in its original location, and verify or revise its description. The term is usually modified to indicate the type or nature of the recovery, such as recovered bench mark, or a recovered triangulation station.

Reduction to sea level A reduction applied to a measured horizontal length on the earth's surface to reduce it to the surface of the sea-level datum of the reference spheroid.

Reference mark A permanent supplementary mark close to a survey station to which it is related by an accurately measured distance and direction, and/or a difference in elevation.

Referencing The process of measuring the horizontal (or slope) distances and directions (azimuths or bearing) from a survey station to nearby landmarks, reference marks, and other objects that can be used in the recovery of the station.

Reflecting prism A prism in which deviation of a light beam is produced by reflection within the prism. Almost all prisms used in photogrammetric instruments are of this type.

Refraction The bending of light rays in passing from one transparent medium into another that has a different index of refraction. The angle of refraction is the angle that the refracted ray makes with the normal to the surface separating the two media.

Refraction, horizontal The lateral effect of terrestrial refraction that affects the observed values of horizontal directions.

Refraction, index of The sine of the angle of incidence divided by the sine of the angle of refraction equals a constant when one of the media is air. The index of refraction can also be explained as the ratio of the velocity of light in one medium to that in another. The indices of glass range from 1 46 to 1 80.

Relief Variations in the elevation of the ground surface, also features of height above a plain or reference datum. On a topographic map, relief is depicted by hachures or shading, or more accurately by contours or by spot elevations or both.

Repetition of angles The accumulation of a series of measures of the same angle on the horizontal circle of a total station. In making these readings the final reading of one measure is used as the initial setting of the next measure of the angle. The observed value of the angle is obtained by dividing the total arc passed over by the number of observations.

Resection

- The graphical or analytical determination of a position, as the intersection of at least three lines of known direction to corresponding points of known position.
- (Surveying) The determination of the horizontal position of a survey station by observed directions from the station to points of known positions. Also, the line drawn through the plotted location of a station to the occupied station.
- (Photogrammetry) The determination of the position and/or altitude of a camera, or the photograph taken with that camera, with respect to the exterior coordinate system.

Restoration The recovery of one or more lines or corner positions, or both, of a prior approved survey; or the replacement of one or more lost corners or obliterated monuments by approved methods, including the substantial renewal of one or more monuments, as required for the purpose of a survey.

Resurvey A retracing on the ground of the lines of an earlier survey, in which all points of the earlier survey that are recovered are held fixed and used as a control. If too few points of the earlier survey are recovered to satisfy the control requirements of the resurvey, a new survey may be made. A resurvey is related directly to an original survey although several resurveys may interpose between them. See also “Resurvey, dependent” and “Resurvey, independent”.

Resurvey, dependent A resurvey for accomplishing a restoration based on the original conditions according to the records. The dependent resurvey is made, first by identifying existing corners and other recognized and acceptable points of control of the original survey, and second, by restoring the missing corners by proportionate measurement in harmony with the original survey. This type of resurvey is used where there is fair agreement between the conditions on the ground and the records of the original survey. Titles, areas, and descriptions should remain unchanged. Contrasted with independent resurvey.

Resurvey, independent A resurvey that is not dependent on the records of the original survey but is intended to supersede them in establishing new land boundaries and subdivisions. It is made in areas generally having both private and public lands represented in the tract to be resurveyed and where the ground evidence of the original survey has become entirely lost or the descriptions of the earlier survey are irreconcilable. An independent resurvey is accomplished in three steps: the out-boundaries of lands subject to resurvey are first reestablished, following the method of dependent resurvey; the private lands are segregated, with due consideration for the bona fide rights of the claimants; and new boundaries and descriptions are established for the remaining public lands. Contrasted with dependent resurvey.

Retracement A term applied to a survey that is made for the purpose of verifying the direction and length of lines, and identifying the monuments and other marks of an established prior survey.

RINEX Receiver INdependent EXchange format A set of standard definitions and formats to promote the free exchange of GPS data and facilities the use of data from any GPS receiver with any software package. The format includes definitions for three fundamental GOS observables: time, phase, and range.

Riprap Large angular stones used to minimize bank erosion.

Rover Any mobile GPS receiver collecting data during a field session. The receiver's position can be computed relative to another, stationary GPS receiver.

S

Scale factor A multiplier for reducing a distance obtained from a map by computation or scaling to the actual distance on the datum of the map. Also, in the state coordinate systems, scale factors are applied to geodetic lengths to obtain grid lengths, or to grid lengths or obtain geodetic lengths.

Screw gate (irrigation) Similar to a slide gate, but generally has a round handle to assist in raising or lowering a gate.

Semi-diameter In making observation of the sun or the moon, the angle should be measured to one edge (or limb) of the body, and the observed angle should be reduced to the center by adding or subtracting the apparent semi-diameter. This quantity may be found in the Nautical Almanac. For the sun, it is approximately 16 minutes of arc but varies about 15 seconds of arc either way.

Set of angles A set of angles consists of two pointings of an instrument. For horizontal angles, a set consists of direct and reverse pointing of the instrument. For zenith angles, a set consists of a left (vertical circle left) and a right (vertical circle right) pointing of the instrument. A meaned set of angles removes systematic instrument adjustment and leveling errors from an observation. Correct operation of instruments requires a minimum of one set of angles for traverse or layout work.

Sidelap The overlap area common to two parallel strips of aerial photographs.

Side shot A reading or measurement from a survey station to locate a point that is not intended to be used as a base for the extension of the survey. A side shot is usually made for the purpose of determining the position of some object that is to be shown on the map.

Single proportionate measurement A method of proportioning measurement in the restoration of a lost corner whose position is determined with reference to alignment in one direction. Examples of such corners are quarter-section corners on the line between two section corners, all corners on standard parallels, and all intermediate positions on any township boundary line. The ordinary field problem consists of distributing the excess of deficiency between two existent corners in such a way that the amount given to each interval bears the same proportion to the whole difference as the record length of the interval bears to the whole distance. After having applied the proportionate difference to the record length of each interval, the sum of the several parts will equal the new measurement of the whole distance.

Slide gate (irrigation) Usually metal and manually slides up and down and located in a head gate. See "Screw Gate".

Spheroid A mathematical figure closely approaching the geoid in form and size and used as a surface of reference for geodetic surveys.

Station A term meaning a point every 100 feet on centerline; in traversing, the name of a traverse point. Basically, the name of a point.

Stereocompilation

- The procedure of producing a map from aerial photographs by means of stereoplotting instruments.
- The map data is produced with stereoplotting instruments.

Stereoscope An optical instrument used for viewing two properly related photographs or diagrams simultaneously to obtain the mental impression of a three-dimensional model.

Stereoscopic principle (photographic mapping) The formation of a single, three-dimensional image by binocular vision of two photographic images of the same terrain taken from different exposure stations. With proper equipment all measurements needed in map construction can be made from this visual model.

Strip adjustment Similar to a block adjustment, but limited to a single strip of photographs.

Subchord Any chord of a circular curve whose length is less than that of the chord adopted for laying out the curve. In a railroad curve, for example, a subchord is a chord less than 100 feet in length. Also, any chord of a circular curve that is less than the long chord between the extremities of the curve.

Subdivision (real estate) A tract of land surveyed and divided into lots for purposes of sale.

Subdivision (USPLS) The subdivision of a township, such as a section, half-section, quarter-section, quarter-quarter or sixteenth-section, or lotting, including the lot, section, township, and range numbers, and the description of the principal meridian to which referred, all according to the approved township plat.

Surveying

- The science and art of making all essential measurements in space to determine the relative position of points and/or physical and cultural details above, on, or beneath the surface of the earth and to depict them in usable form, or to establish the position of points and/or details. Also, the actual making of a survey and the recording and/or delineation of dimensions and details for subsequent use.
- The acquiring and/or accumulation of qualitative information and quantitative data by observing, counting, classifying, and recording according to need. Examples are traffic surveying to determine the type, number, speed, relative positions, and origin and destination of vehicles; and soil surveying to classify soil by type and measure and delineate their boundaries.

Surveying, geodetic That branch of the art of surveying in which account is taken of the figure and size of the earth. In geodetic surveying, prescribed precision and accuracy of results are obtained through the use of special instruments and field methods and equations based on the geometry of a mathematical figure approximating the earth in form and size.

Surveying, land Land surveying is the art and science of:

- re-establishing cadastral surveys and land boundaries based on documents of record and historical evidence;
- planning, designing and establishing property boundaries; and
- certifying surveys as required by statute or local ordinance such as subdivision plats, registered land surveys, judicial surveys, and space delineation.

Land surveying can include associated services such as mapping and related data accumulation; construction layout surveys; precision measurements of length, angle, elevation, area and volume; horizontal and vertical control systems; and the analysis and utilization of survey data.

Surveying, plane A branch of the art of surveying in which the surface of the earth is considered a plane surface. For small areas, precise results may be obtained with plane-surveying methods, but the accuracy and precision of such results will decrease as the area surveyed increases in size.

Surveying, route For locating, designing, and constructing a railroad, highway, canal, pipeline, transmission line, or other linear facility. Route surveying comprises all reconnaissance surveys, the preliminary survey, the location survey, and surveys made during construction.

T

Tangent

- A straight line that touches a given curve at one and only one point and does not intersect it.
- In route alignment, that part of alignment from one PI to the next PI; the distance from the PT of one curve to the PC of the next curve; or the distances from the PC to PI and PI to PT of a curve.

Taping The operation of measuring distances on the ground with a tape or chain. Formerly the words “chaining” and “taping” were used synonymously, but the word “taping” is now preferred for all surveys except those of the public-land system. For the latter, because of historical and legal reasons, the term “chaining” is preferred. See also “Chaining”.

Taping (breaking tape) A method of taping on slopes whereby all measurements are made with a part of the tape held horizontally.

Taping, standard tension The tension or pull at which a tape was standardized.

Target

- Any object or point toward which something is directed.
- An object that reflects a sufficient amount of a radiated signal to produce an echo signal on detection equipment.

Target, photographic A pre-marked image point used to control the photography for mapping purposes or to extend control by photogrammetric methods. A target is centered over a point that would otherwise not be visible on a photo. A target generally is black and white and may be made of various materials (for example: paint, plastic, cloth or lumber). Also called signals or panels. The preferred term is photographic target.

Temporary Bench mark A semi-permanent object, where the elevation above or below an adopted datum is known. Usually designated as a TBM.

Thalweg The line following the lowest part of a valley. Generally used for the location of the lowest points in a river or creek.

Thence In surveying, and in descriptions of land by courses and distances, this word, preceding each course given, imports that the following course is continuous with the one before it.

Theodolite A precision surveying instrument consisting of an alidade with a telescope. It is mounted on an accurately graduated circle and is equipped with necessary levels and reading devices. Sometimes, the alidade carries a graduated vertical circle. There are two general classes of theodolites: direction theodolites and repeating theodolites.

Theodolite, directional instrument A theodolite in which the graduated horizontal circle remains fixed during a series of observations. The telescope is pointed on a number of signals or objects in succession, and the direction of each is read on the circle, usually by the use of micrometer microscopes. In measuring horizontal angles with a direction instrument, angles are not repeated (accumulated) on the circle, but precision and accuracy are obtained by having the circle of high quality, by using precision methods of reading the circle, and by shifting the circle between sets so that each direction is measured on a number of different parts of the circle. Direction instruments are used almost exclusively in first-order and second-order triangulation.

Theodolite, repeating instrument A theodolite so designed that successive measures of an angle may be accumulated on the graduated circle, and a final reading of the circle, which represents the sum of the repetitions, may be made. The observed value of the angle is obtained by dividing the total arc passed through in making the series of observations by the number of times the angle has been observed. The total arc passed through may include several complete circuits of the circle, which must be added to the circle reading before making the division. The repeating theodolite is also termed a repeating instrument. Theoretically, it is an instrument of great precision, but in its mechanical operation it does not give results as satisfactory as the directional instrument.

Tilt (photogrammetry) The angle at the perspective center between the photograph perpendicular and the plumbline (or other exterior reference direction); also, the dihedral angle between the plane of the photograph and the horizontal plane. The direction of tilt is expressed by swing (when referred to the axes of the photograph) or azimuth (when referred to the exterior coordinate system). In aerial photography, tilt may be separated into its component angles, referred to the fiducial axis, with the x-axis being the one more nearly in the direction of flight. In aerial-camera orientation, a positive x tilt results from the left wing of the aircraft being lowered, displacing the nadir point in the positive y direction. Similarly a positive y tilt results from the nose of the aircraft being lowered, displacing the nadir point in the positive x direction.

Total stations Total stations are classified as automatic or manual. Field procedures are different for the two classes.

Traverse A method of surveying in which lengths and directions of lines between points on the earth are obtained by or from field measurements, and used in determining positions of the points. A survey traverse may determine the relative positions of the points that it connects in series, and if tied to control stations on an adopted datum, the positions may be referred to that datum. Survey traverses are classified and identified in a variety of ways: according to methods used, as astronomical traverse; according to quality of results, as first-order traverse; according to purpose served, as geographical-exploration traverse; and according to form as closed traverse, etc.

Traverse, closed A survey traverse that starts and ends upon the same station, or upon stations whose relative positions have been determined by other surveys of equal or higher order of accuracy.

Traverse, open A survey traverses that begins from a station of known or adopted position, but does not end upon such a station. Also termed an open-end traverse.

Triangulation A method of surveying in which the stations are points on the ground at the vertices of a chain or network of triangles. The angles of the triangles are measured instrumentally and the sides are derived by computation from selected sides or bases, the lengths of which are obtained by direct measurement on the ground or by computation from other triangles. A triangulation system of limited width (generally that of one triangle), designed to progress in a single general direction, is designated arc triangulation, and the chain of triangles (or polygons composed of abutting or overlapping triangles) is called a triangulation arc. A network of triangulation designed to cover an area with abutting or overlapping triangles is designated area triangulation, and the resulting configuration is called a triangulation net.

Triangulation (photogrammetry) see aerotriangulation.

Tribrach The three-arm base of a surveying instrument that carries the foot-screws used in leveling the instrument.

Trilateration A method of surveying wherein the lengths of the triangle sides are measured, usually by electronic methods, and the angles are computed from the measured lengths. Compare with triangulation.

True value That value of quantity that is completely free from blunders and errors. Since the errors to which physical measurements are subject cannot be known exactly, it follows that the true value of a quantity cannot be known with exactness. In survey work, the most probable value is used as best representing the true value of the quantity.

Turnout (irrigation) A structure located above a check structure and is used for routing water out of the main irrigation channel.

Turning point (TP) In differential leveling, a point, the height of which is determined before the leveling instrument is moved, used to determine the height of the instrument after resetting.

U

Underground mark A surveying mark set and plumbed below the center of a surface mark. It is separated from the surface mark in order to preserve the station in case of destruction of the surface mark.

V

Vertical angle An angle between two intersecting lines in a vertical line.

Vertical control Established bench marks.

Vertical control point A targeted or photo ID point that has only vertical or elevation control established. In order to properly control photographs for highway mapping purposes, it is necessary to have more vertical control points than horizontal control. When leveling to vertical control points, three elevations at the point need to be recorded, even if they are the same. First, the elevation of the monument set at the point is required. This is the point through which the level circuit will be closed. Second is the elevation of the ground at the center of the target (the mean elevation of a one-foot diameter circle around the point). The third is the elevation of the center of the target. This is important when the point and the ground are in a depression which the target bridges.

Viewfinder (aerial photogrammetry) A device similar to a camera but with the ground glass in the focal plane of the lens. The viewfinder is mounted vertically in the floor of an airplane for the purpose of viewing the landscape and determining when photographs should be taken. It is graduated to determine the spacing between photographs necessary to obtain the desired overlap. Because the viewfinder is aligned with the true direction of flight, the angle of crab between the longitudinal axis of the plane and the direction of flight can be obtained.

W

WAAS Wide Area Augmentation System is a FAA funded system of equipment and software that augments GPS. The WAAS provides a satellite signal for WAAS users to support enroute and precision approach aircraft navigation.

Watershed The area contained within a drainage divide above a specified point on a stream. In water-supply engineering it is termed a watershed, and in river-control engineering it is termed a drainage area, drainage basin, or catchment area.

Weight (surveying) The relative reliability (or worth) of a quantity as compared with other values of the same quantity. If one value of a quantity has a weight of 2, and another value of the same quantity has a weight of 1, the first value is worth twice the second value, and a mean value would be obtained by taking a weighted mean twice the first value plus once the second value, the sum being divided by 3.

Wiggling in A survey procedure used when it is necessary to establish a point, exactly on line between two control points, neither of which can be occupied. It is essentially a trial-and-error technique where repeated fore and back readings are taken and the instrument is shifted after each pair of readings until exactly in line with the stations. Also called ranging in.

Wing point A vertical control point, either targeted or identified as a picture point, usually placed in the corners of the neat model.

Witness mark A material mark placed at a known distance and direction from a property corner, instrument, or other survey station, to aid in its recovery and identification. In surveying, a witness mark is established as an aid in the recovery and identification of a survey station, or other point to which it is a witness. A mark, which is established with such precision and accuracy that it may be used to restore or take the place of the original station is more properly, called a reference mark in control surveys, and a witness corner in land surveys. Also called witness post or witness stake.

Witness point (USPLS) A monumented station on a line of the survey.

Witness post Delineator post driven three feet southwest of basic ground control survey stations.

Witness sign A metal fence post or composite fiber post with a “Do Not Disturb Survey Monument” sign attached. The witness sign is set close to a survey control point to aid in its protection and recovery.

X

X, Y, and Z coordinates Three separate coordinate values that are referenced to a known or adopted datum. The “X” value is an east-west or longitudinal coordinate. The “Y” value is a north-south or latitudinal coordinate value. The “Z” value is an elevation value. When coordinates are computed or listed, it is common practice to list the “Y” (north) value first, next the “X” (east) value, and then the “Z” (elevation) value.

Z

Zenith Where the point produced by a plumb line projected up from the observer meets the Celestial Sphere.

Zenith angle An angle measured from the zenith (straight up) to a point. In modern survey instruments, the zenith angle replaces the vertical angle and is used to reduce slope measurements to horizontal and vertical differences. The use of zenith angles correctly signs the value of the vertical difference for computer computation.

Abbreviations

The following are abbreviations typically used in **conventional** survey notes

Adjusted	ADJ
Ahead	AHD or AH
And&	
Aluminum Cap	AC
At	@
Avenue	AVE
Average	AVG
Azimuth	AZ
Back	BK
Backsight	BS
Bearing	BRG
Begin	BEG
Bench Mark	BM
Bottom	BOT
Bottom of Bank	BOB
Bottom of Slope	BOS
Boulevard	BLVD
Boundary	BNDRY
Brass Cap	BC
Bridge	BR
Bridge End	BE
Building	BLDG
Bureau of Land Management	BLM
Calculated	CALC
Cast Iron Pipe	CIP
Catch Basin	CB
Catch Point	CP
Cement Treated Base	CTB
Centerline	C/L
Centimeter	CM or cm
Chain	CH
Chain Link Fence (w/ height)	CL-4F,5F,6F
Chord	CHD
Clean Out	CO
Closing Corner	CC
Cloudy	CLDY
Concrete	CONC

Concrete Block Wall	CBW
Concrete Nail	CN
Conduit (specify type)	COND (TEL)
Construction	CONST
Control Number	CN
Control of Access	C/A
Corner	COR
Corrugated Metal Pipe	CMP
Corrugated Steel Pipe	CSP
County	CO
Creek	CR or CK
Crossing	XING
Cross Section	XSEC
Culvert	CULV
Curb	CB
Curb and Gutter	C&G
Current Water Surface Elevation	WSE
Curve to Spiral	CS
Cut	C
Daylight	DL
Deflection	DEFL
Degree of Curvature	D
Description	DESC
Destroyed	DEST
Detour	DET
Diameter	DIA
Difference in Elevation	DE
Distance	DIST
District	DIST
District Construction Engineer	DCE
Ditch	DT
Double	DBL
Down Drain	DD
Drain	DR
Drill Hole	DH
Drive	DR
Driveway	DRWY
Drop Inlet	DI
East	E
East Bound	EB
Easterly	ELY
Edge of Gutter	EG

Edge of Oil	EO
Edge of Pavement	EP
Edge of Shoulder	ES
Edge of Traveled Way	ETW
Electronic Distance Measurement or Measurer	EDM or EDMI
Elevation	EL or ELEV
End Wall	EW
Engineering Project Manager	EPM
Equation	EQ
Existing	EX
External	EXT
Fahrenheit	F
Fence	FN or FX
Fence Post	FP
Feet	FT or ‘
Field Book	FB
Fill	F
Finish Grade	FG
Finish Grade Stake	FGS
Fire Hydrant	FH
Flood Control	FC
Flow Line	FL
Flush	FL
Foot	FT or ‘
Footing	FTG
Foresight	FS
Found	FND
Foundation	FDN
Frontage Road	FR RD
Galvanized	GALV
Galvanized Steel Pipe	GSP
Gas Line	GL
Gas Valve	GV
Global Positioning System	GPS
General Land Office	GLO
Grade Separation	GR SEP
Grid	GRD
Ground	GRND
Guard Rail	GR
Gutter	GTR
Headwall	HDWL
Height	HT

Height of Instrument	HI
Highway	HWY
High Water	HW
Hinge Point	HP
Horizontal	HORIZ
Hub & Tack	H&T
Inch	IN or “
Inside Diameter	ID
Instrument	INSTR
Interchange	INTCH
Intersection	INT
Invert	INV
Iron Pin	IP
Irrigation Pipe	IRR P
Joint Use Pole	JP
Junction	JCT
Kilometer	KM or km
Land Surveyor	LS
Lane	LN
Left	LF or LT
Length of Circular Curve	L_C or L
Length of Spiral Curve	L_S
Length of Vertical Curve	L
Link	lk
Location Hydraulic Study Report	LHSR
Long Chord	LC
Manhole	MH
Marker	MKR
Maximum	MAX
Measured	MEAS
Median	MED
Meter	M or m
Mid Ordinate	MO
Mid-Point of Curve	MPC
Mile	MI
Mile Post	MP
Millimeter	MM or mm
Minute	MIN or ‘
Montana Department of Transportation	MDT
Nail	NL
National Geodetic Survey	NGS
National Oceanic & Atmospheric Administration	NOAA

Natural Gas Line	NG
Normal Water Elevation	NW EL
North	N
North Bound	NB
Northerly	NLY
Number	# or NO
Offset	O/S or O
Old Ground	OG
On Centers	OC
Original Ground	OG
Outside Diameter	OD
Overhang	OH
Overhead Crossing	OC
Overhead	OH
Page(s)	PG or PGS
Parker-Kalon Nail	PK
Parts per Million	PPM or ppm
Party Chief	PC
Pavement	PVMT
Penny (i.e.: 60d nail)	d
Perforated Metal Pipe	PMP
Pipe	P
Place	PL
Plant Mixed Surface	PMS
Plastic	PLAS
Point of Curvature	PC
Point of Compound Curvature	PCC
Point of Intersection	PI
Point on Curve	POC
Point on Line	POL
Point on Semi-Tangent	POST
Point on Tangent	POT
Point on Vertical Curve	POVC
Power Pole	PP
Present Traveled Way	PTW
Principal Meridian	PM
Private	PVT
Professional Engineer	PE
Profile Grade	PG
Project	PROJ
Project Manager	PM
Projected	PROJ

Property Line	PL
Pull Box	PB
Punch Mark	PM
Radial	RAD
Radius	R
Radius Point	RP
Railroad or Railway	RR
Range	R
Record	REC
Reference	REF
Reference Monument	RM
Reference Point	RP
Reference Post	RP
Reinforced Concrete Box	RCB
Reinforced Concrete Pipe	RCP
Retaining Wall	RET W
Right	RT
Right of Way	R/W
Road	RD
Roadway	RDWY
Rounding	RN
Route	RT
Searched For Not Found	SFNF
Second	SEC or “
Section	S
Sewer Line (Sanitary)	SS
Sheet	SHT
Shoulder	SH
Sidewalk	SW
Slope Stake	SLP STK
South	S
South Bound	SB
Southerly	SLY
Spike	SPK
Spiral to Curve	SC
Spiral to Tangent	ST
Stake	STK
Staked	STKD
Standard	STD
Stand Pipe	SP
State Plane Coordinate	SPC
Station	STA

Steel	STL
Steel Pipe, High Pressure	SPHP
Steel Sectional Plate Pipe	SSPP
Storm Drain/Sewer	SD
Street	ST
Structure	STR
Subdivision	SUBD
Subgrade	SG
Surfacing	SURF
Survey	SURV
Tack	TK
Tangent	TAN
Tangent Length of Curve	T
Tangent to Spiral	TS
Telephone Cable	TEL C
Telephone Pole	TEL P
Temperature	TEMP
Temporary Bench Mark	TBM
Top Back of Curb	TBC
Top of Bank	TOB
Topographic	TOPO
Township	T
Tract	TR
Transmission Tower	TT
Traverse	TRAV
Turning Point	TP
Uniform Project Number	UPN
U S Coast & Geodetic Survey	USC&GS
U S Corps of Engineers	USCE
U S Forest Service	USFS
U S Geological Survey	USGS
U S Public Land Survey	USPLS
Vent Pipe	VP
Vertical Curve	VC
Vitrified Clay Pipe	VCP
Warped or Variable Slope	WS
Water Line	WL
Water Service	WS
Water Table	WT
Water Valve	WV
Welded Steel Pipe	WSP
West	W

West Bound	WB
Westerly	WLY
Wing Point	WP
Wing Wall	WW
With	W/
Witness Corner	WC
Woven Wire	WW
Yard	YD

Surveyor's Measures And Conversions

Constants

- Pi, π The number that denotes the ratio of the circumference of a circle to its diameter = 3.1415926536
- Radian The central angle of a circular arc that is equal in length to the radius of the arc
- $180° \div \pi = 57.29577951°$ = one radian
- Mean Radius of the Earth = ±20,906,000 feet or 6,372,000 meters

Temperature Conversions

- Celsius to Fahrenheit = (9/5 C) + 32
- Fahrenheit to Celsius = 5/9 (F - 32)

Abbreviations

- Linear (US)

Chain	ch
Link	lk
Mile	mi
Foot	ft or ‘
Inch	in or "
Yard	yd

- Linear (Metric)

Centimeter	cm
Meter	m
Micron	µ
Millimeter	mm
Kilometer	km

- Area (US and Metric)

Acre	A
Hectare	ha
Square Feet	Sq Ft

- Pressure

Millibar	mb	hecto
Pascal	hPa	

Linear Relationships (US)

1 lk = 7.92 in or 0.66 ft
80 ch = 1 mi
3 ft = 1 yd
1 ch = 66 ft or 100 lk
12 in = 1 ft
5280 ft = 1 mi

Linear Relationships (Metric)

0.001 m = 1 mm
10 mm = 1 cm
1 m= 100 cm
0.01 m = 1 cm
1,000 m = 1 km

Area (US)

1 A = 43,560 sq ft
1 sq mi = 640 A

Area (Metric)

10,000 sq m = 1 ha

Linear Conversions

1 in = 0.0254 m = 25.4 mm (US Survey foot)
1 ft (US Survey foot) = 12/39.37 = 0.30480061 m
1 ft (International foot) = 0.3048
1 mi = 1609.347 m (US Survey foot) or 1609.344 m (International foot)
1000 m = 0.62137 mi

Currently state statutes require the use of the international foot when converting state plane coordinates from meters to feet. The US Survey foot is generally used to convert total station distances from meters to feet.

Area Conversions

1 ha = 2.4711 A
1 A = 0.40469 ha

Pressure Conversions

1 in of mercury = 25.4 mm of mercury
1 mm mercury = 1.333333333 hPa

Appendix D
Basic Trigonometry

Appendix D provides basic trigonometric formulas for solving right triangles and oblique triangles.

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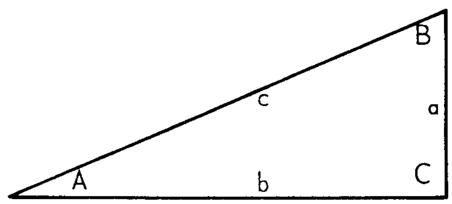
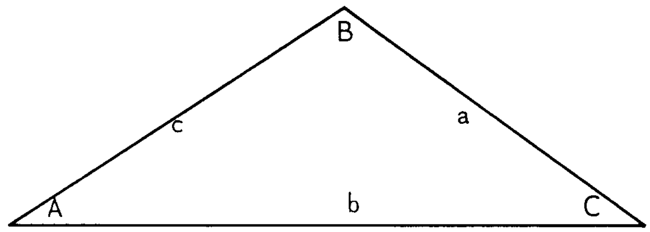
Right Triangles



C = 90 degrees

Given	To Find	Formulas
a, b	c	$\sqrt{a^2 + b^2}$
	A	$\tan A = \frac{a}{b}$ or $\cot A = \frac{b}{a}$
	B	$\tan B = \frac{b}{a}$ or $\cot B = \frac{a}{b}$
	Area	$\frac{a\,b}{2}$
a, c	b	$\sqrt{c^2 - a^2}$
	A	$\sin A = \frac{a}{c}$
	B	$\cos B = \frac{a}{c}$ or $90° - A$
A, a	b	$\frac{a}{\tan A}$
	c	$\frac{a}{\sin A}$
	B	$90° - A$
A, b	a	$b (\tan A)$
	c	$\frac{b}{\cos A}$
	B	$90° - A$
A, c	a	$c \sin A$
	b	$c \cos A$
	B	$90° - A$

Oblique Triangles



Law of Sines: $\frac{a}{\sin A} = \frac{b}{\sin B} = \frac{c}{\sin C}$

Law of Cosines: $a^2 = b^2 + c^2 - 2bc \cos A$

or $\cos A = \frac{b^2 + c^2 - a^2}{2bc}$

Given	To Find	Formulas
a, b, c	A, B, C Using “s”	Law of Cosines, or $\sin \frac{1}{2} A = \sqrt{(s - b)(s - c)/bc}$; $\cos \frac{1}{2} A = \sqrt{s(s - a)/bc}$ $\sin A = \frac{2\sqrt{s(s - a)(s - b)(s - c)}}{bc}$ Note: For angles B & C make appropriate substitutions in these formulas. Note: The value “s” = $\frac{1}{2}(a + b + c)$
	Area	$\sqrt{s(s - a)(s - b)(s - c)}$
a, A, B	b C c	Law of Sines $180° - (A + B)$ Law of Sines
a, b, A	B C c	Law of Sines $180° - (A + B)$ Law of Sines
a, b, C	c A B Area	Law of Cosines $\tan A = \frac{a \sin C}{b - (a \cos C)}$ $180° - (A + C)$ $\frac{1}{2} ab \sin C$
ABC, a	Area	$\frac{a^2 (\sin B)(\sin C)}{2 \sin A}$

Appendix E

Curves

The formulas for computing the various curves used by the Department are presented in Appendix E.

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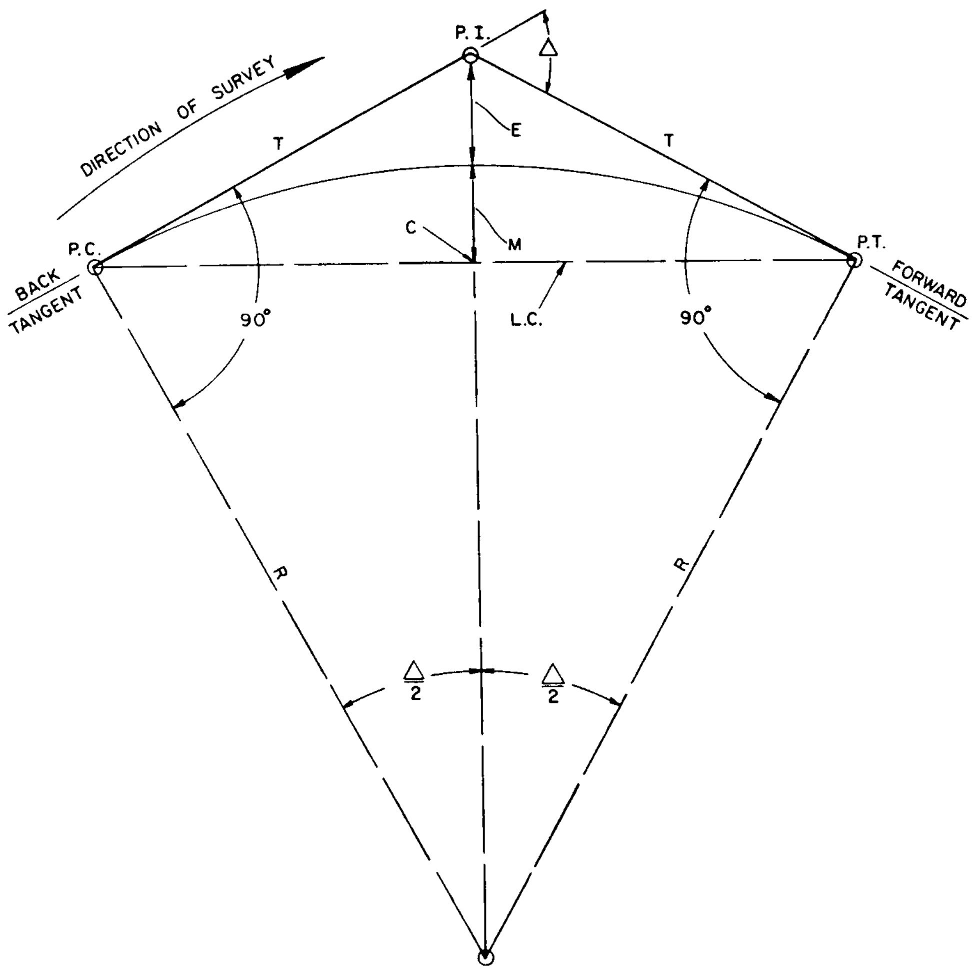
Circular Curves



P. I. = Point of Intersection
P. C. = Point of Curvature
P. T. = Point of Tangency
Δ = Deflection angle between the tangents
T = Tangent distance
E = External distance
R = Radius of the circular curve
M = Middle ordinate
L. C. = Long chord (Distance between P. C. and P. T.)
C = Midpoint of the long chord
D = Degree of curvature

General Formulas for Arc Definition

$T = R \tan \frac{\Delta}{2}$

$D = \frac{5729.578}{R}$

$L.\ C. = 2\ R \sin \frac{\Delta}{2}$

$E = T \tan \frac{\Delta}{4}$ or $E = R \text{ exsec} \frac{\Delta}{2}$ or $E = R \sec \frac{\Delta}{2} - R$

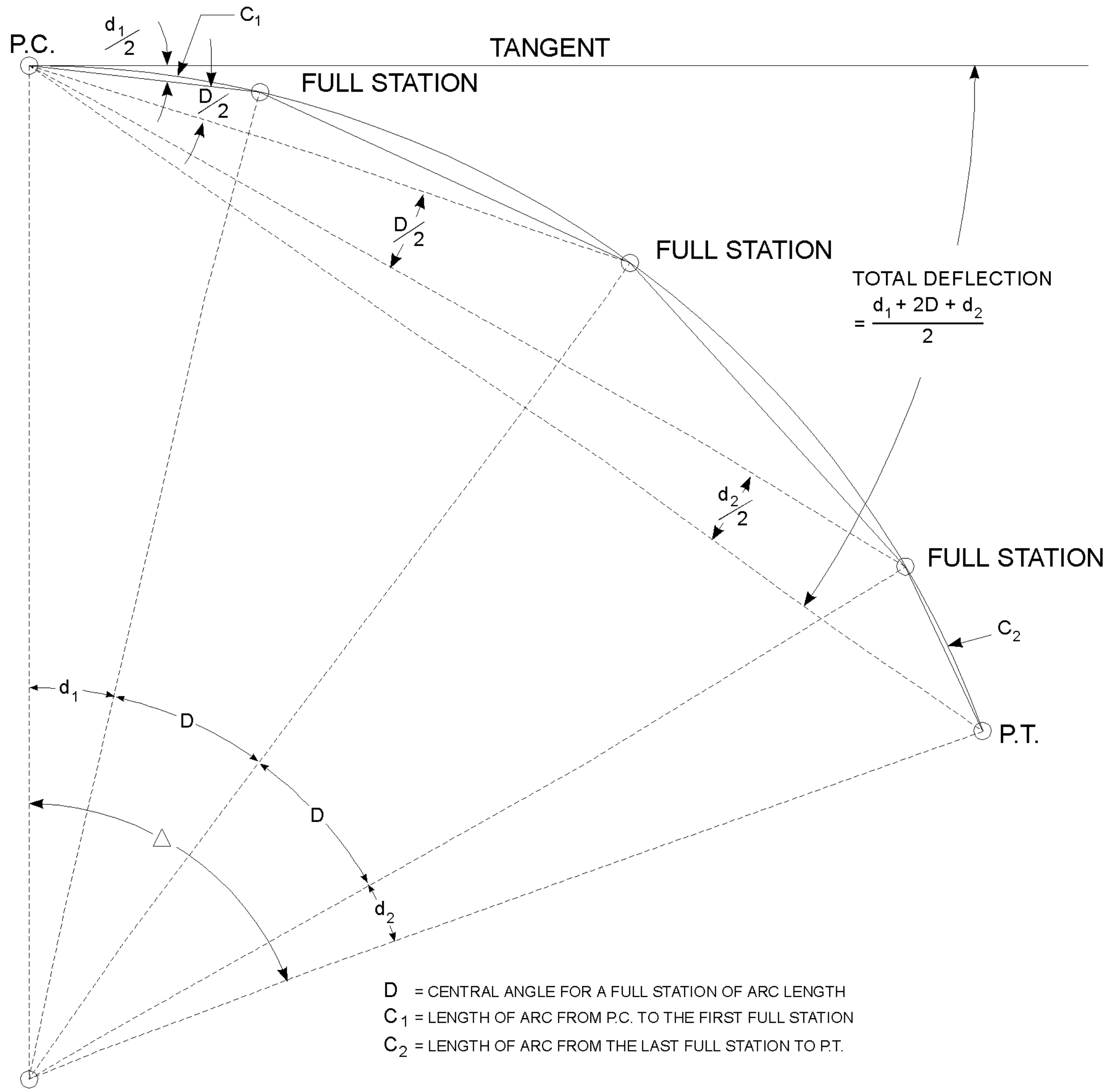
$M = E \cos \frac{\Delta}{2}$ or $M = R \left(1 - \cos \frac{\Delta}{2}\right)$

Length of curve, L, = $\frac{100\ \Delta}{D}$ when Δ and D are in minutes

Locating the P. C. and P. T.

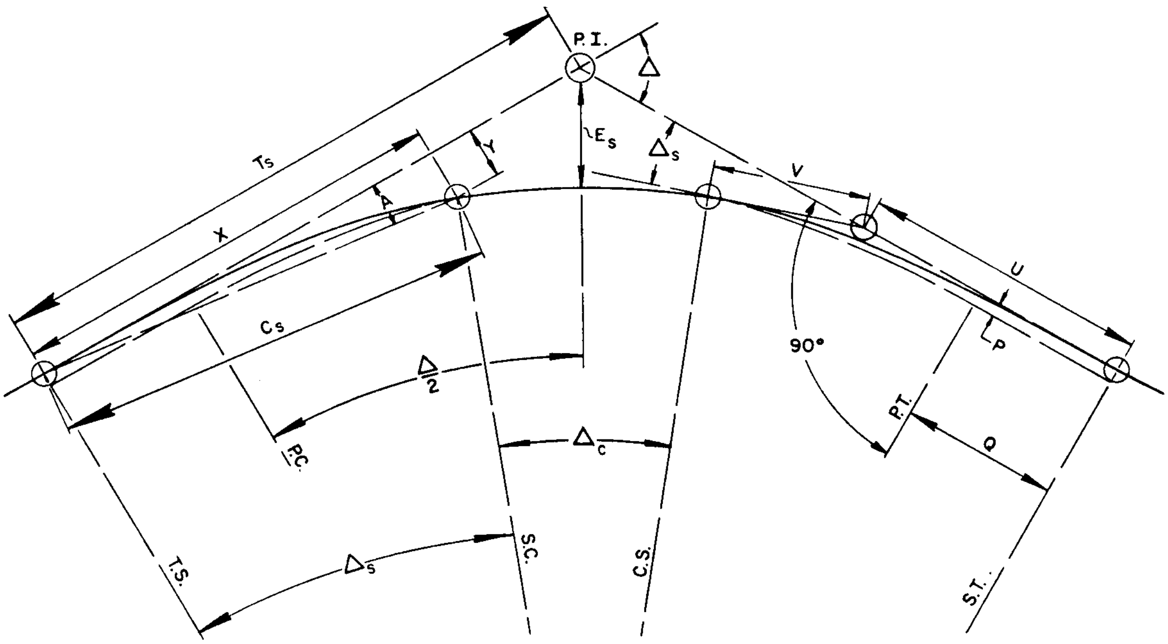
Station of P. C. = Station of P. I. – T
Station of P. T. = Station of P. C. + L

Simple Curve Computation by Deflection Angle



1. Get the P. C. and P. T. stations and the value of D in decimals of a degree, by using the general formulas for circular curves, from the plans, or computer run.
2. The chord for any arc length: 2 R sin (deflection angle)
3. The deflection angles in degrees, for full station arc length equals $\frac{D}{2}$.
4. The deflection angle, in decimal degrees, for any arc length: 0.005 D (arc length)
5. A running total of station to station deflections gives the total deflection angle from the P. C.
6. Computation Check: The total deflection angle to the P. T. must equal $\frac{\Delta}{2}$

Spiral Curve Transitions



L_S = Length of spiral
D = Degree of curvature of the circular curve
T_S = Tangent distance
Δ = Deflection angle between the tangents
Δ_S = Spiral angle
Δ_C = Central angle between the S. C. and C. S.
E_S = External distance
C_S = Long chord
U = Long tangent
V = Short tangent

X = Spiral coordinate (Abscissa)
Y = Spiral coordinate (Ordinate)
Q = Simple curve coordinate (Abscissa)
P =Simplecurve coordinate (Ordinate)
Δ = Deflection angle of spiral curve
K = Constant of increasing curvature
T. S. = Tangent to spiral
S. C. = Spiral to curve
C. S. = Curve to spiral
S. T. = Spiral to tangent

Spiral Curve Formulas

$\Delta_S = \frac{D\,L_S}{200}$ $\qquad D = K\,L_S$ $\qquad \Delta_S = \frac{K\,L_S^2}{200}$

To Calculate Tangent Distances of a Simple Curve with Equal Spirals

$$T_S = (R + P)\tan\frac{\Delta}{2} + Q$$

$$T_S = T + Q + P\tan\frac{\Delta}{2}$$

$$E_S = \frac{(R + P)}{\cos\frac{\Delta}{2}} - R$$

$$E_S = (R + P)\,\text{exsec}\,\frac{\Delta}{2} + P$$

$$E_S = E + \frac{P}{\cos \frac{\Delta}{2}}$$

$$\Delta_C = \Delta - 2\,\Delta_S$$

To Calculate Tangent Distances of a Simple Curve with Unequal Spirals

$$T_{S_1} = \frac{(R + P)_2}{\sin \Delta} - (R + P)_1 \cot \Delta + Q_1$$

$$T_{S_2} = \frac{(R + P)_1}{\sin \Delta} - (R + P)_2 \cot \Delta + Q_2$$

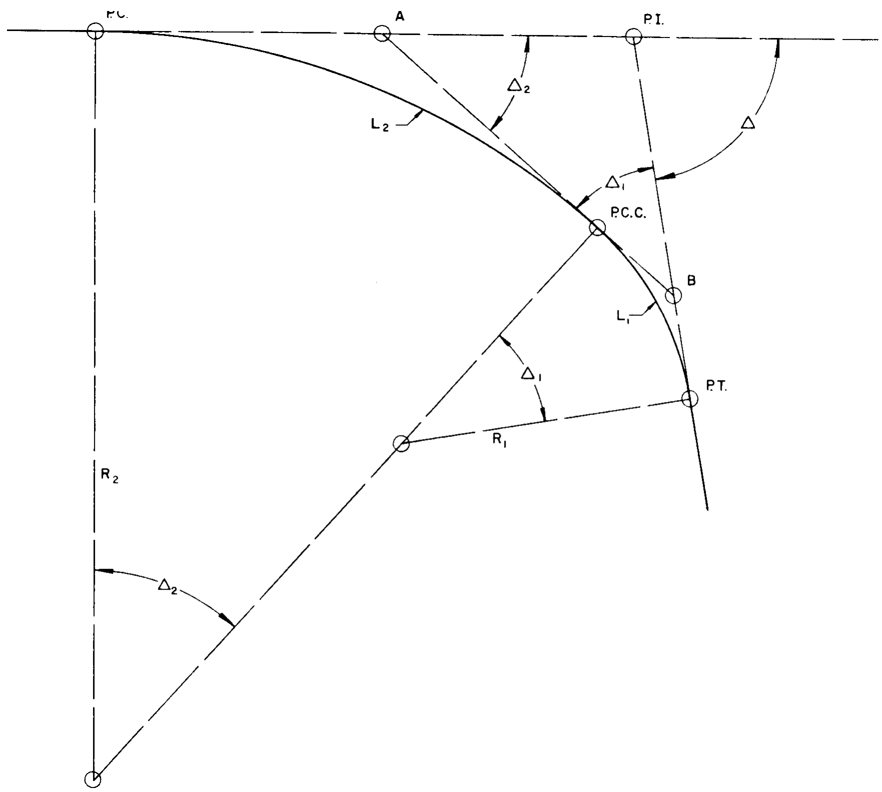
For Deflection Angles to Intermediate Points on Spiral:

A (in minutes) = 10 K S^2, where S = distance in stations to point. This value may have to be adjusted downward. Note that the one-tenth minutes of the tabulated defection angles form a wave-like pattern (equivalent to the above formula) until they fall off near the bottom of the table. Adjustments for intermediate points may be interpolated.

To set stakes with the instrument, set up at a point on the spiral to take advantage of the fact that the spiral is laid out on a system of coordinates. First, set the instrument plate at zero parallel to the tangent of the spiral. This is done by sighting back on the T. S. with “A” for the instrument set-up point on the plates; or by bringing the instrument tangent to the curve with Δ_S for the instrument set-up point on the plates. Then the difference between coordinates of points to be set and the instrument is the deflection angle:

$$\text{Deflection angle} = \tan^{-1} \frac{Y_1 - Y_0}{X_1 - X_0}$$

Compound Curve Computations



Given Δ, R_1, R_2, L_1, L_2 $\qquad \Delta = \Delta_1 + \Delta_2$

Distance to "A" from P.C. equals $R_2 \tan \frac{\Delta_2}{2}$

Distance to "B" from P. T. equals R_1 tan

Curve is laid out as two adjacent simple curves: one having a radius "R_2" with its P. I. at "A" and the other having a radius of "R_1" with its P. I. at "B."

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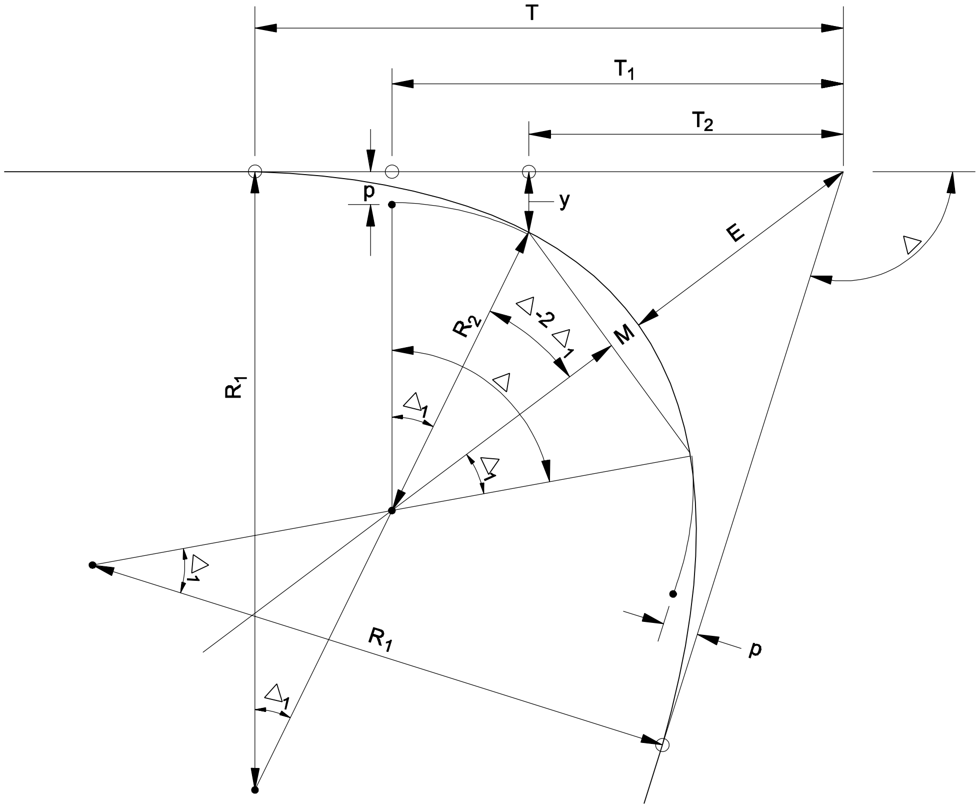
Three-Center Curves



CURVE FORMULA

1. $T_1 = (R_2 + p)\tan\dfrac{\Delta}{2}$

2. $\Delta_1 = \cos^{-1}\left[\dfrac{R_1 - R_2 - p}{R_1 - R_2}\right]$

3. $T = T_1 + (R_1 - R_2)\sin\Delta_1$

4. $T_2 = T_1 - R_2 \sin\Delta_1$

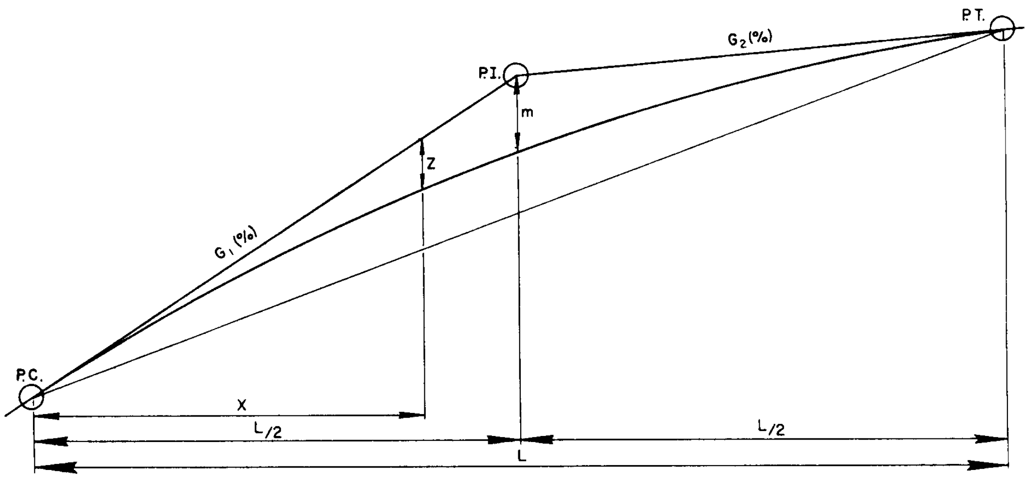
5. $E = \dfrac{R_2 + p}{\cos(\Delta/2)} - R_2$

6. $M = R_2 - \left[R_2\cos(\Delta/2 - \Delta_1)\right]$

7. $y = (R_2 + p) - R_2\cos\Delta_1$

Note: "p" is the offset location between the interior curve (extended) to a point where it becomes parallel with the tangent line.

Symmetrical Vertical Curves



m = Mid-ordinate, in feet
Z = Any tangent offset, in feet
L = Horizontal length of vertical curve, in stations
X = Horizontal distance from P. C. or P. T. to any ordinate “Z,” in stations
G_1 & G_2 = Rates of grade, expressed algebraically, as a percentage

All expressions to be calculated algebraically.

Elevation of P. I. = Elevation of P. C. + $G_1 \frac{L}{2}$

Elevation of P. T. = Elevation of P. C. + $(G_1 + G_2) \frac{L}{2}$

m $= \frac{(G_2 - G_1)\, L}{8} = \frac{1}{2} \left(\frac{\text{Elev. of P.C.} + \text{Elev. of P.T.}}{2} - \text{Elev. of P.I.} \right)$

For offset “Z” at distance “X” from P. C. or P. T.:

$$Z = m \left(\frac{X}{L/2} \right)^2 \text{ or } Z = \frac{X^2 (G_2 - G_1)}{2L}$$

For slope “S” of a line tangent to any point on the vertical curve at an “X” distance measured from the P. C.:

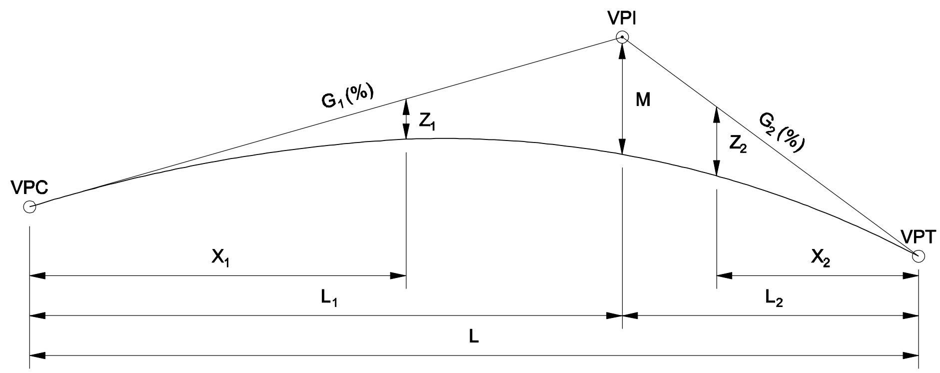
S $= G1 - \left[X \left(\frac{G_1 - G_2}{L} \right) \right]$

Calculating High or Low Point on Curve

$X_T = \frac{L\,G}{G_1 - G_2}$ Where “X_T” equals the horizontal distance, in stations, from the P. C. to the high or low point on the curve.

Elevation of high or low point on curve equals: Elevation of P. C. $- \frac{1}{2} \left(\frac{L\, G_1^2}{G_2 - G_1} \right)$

Unsymmetrical Vertical Curves



M = Offset from the VPI to the curve (external distance), m

Z = Any tangent offset, m

L = Horizontal length of vertical curve, m

L_1 = Horizontal distance from VPC to VPI, m

L_2 = Horizontal distance from VPI to VPT, m

X = Horizontal distance from VPC or VPT to any ordinate "Z," m

G_1 & G_2 = Rates of grade, expressed algebraically, percent

NOTE: ALL EXPRESSIONS TO BE CALCULATED ALGEBRAICALLY
(Use algebraic signs of grades; grades in percent.)

1. Elevations of VPC and VPI:

$$ELEV.\ OF\ \ VPC = ELEV.VPI - G_1\left(\frac{L_1}{100}\right)$$

$$ELEV.\ OF\ \ VPT = ELEV.VPI + G_2\left(\frac{L_2}{100}\right)$$

2. For the elevation of any point "X" on the vertical curve:

$$CURVE\ ELEV. = TAN.ELEV. + Z$$

Where:

Left of VPI (X_1 measured from VPC):

(a) $$TAN.ELEV. = VPC\ ELEV. + G_1\left(\frac{X_1}{100}\right)$$

Right of VPI (X_2 measured from VPT):

(a) $$TAN.ELEV. = VPT\ ELEV. - G_2\left(\frac{X_2}{100}\right)$$

(b) $Z_1 = X_1^2\left(\frac{L_2}{L_1}\right)\left(\frac{G_2 - G_1}{200\ L}\right)$ (b) $Z_2 = X_2^2\left(\frac{L_1}{L_2}\right)\left(\frac{G_2 - G_1}{200\ L}\right)$

3. Calculating High or Low Point on Curve:

Note: Two answers will be determined by solving the equations below. Only one answer is correct. The incorrect answer is where $X_T > L_1$ on the left side of the VPI or where $X_T > L_2$ on the right side of the VPI.

a. Assume high or low point occurs left of VPI to determine the distance, X_T, from VPC:

$$X_T = \frac{L_1}{L_2}\left[\frac{G_1 L}{(G_1 - G_2)}\right]$$

Note: Does $X_T > L_1$? If yes, this answer is incorrect and the high or low point is on the right side of the VPI. (Go to step d. to solve for the high or low point elevation.) If no, then this is the correct answer and proceed with steps b. and c. below.)

b. To determine high or low point stationing (where $X_T < L_1$):

$$STA_{\ HIGH\ OR\ LOW\ POINT} = VPC\ STA. + X_T$$

c. To determine high or low point elevation on vertical curve (when $X_T < L_1$):

$$ELEV_{HIGH\ OR\ LOW\ POINT} = ELEV.VPC - \frac{L_1}{L_2}\left[\frac{LG_1^2}{(G_2 - G_1)200}\right]$$

d. If $X_T > L_1$ from step a., the high or low point occurs right of the VPI. Determine the distance X_T from the VPT:

$$X_T = \frac{L_2}{L_1}\left[\frac{G_2 L}{(G_2 - G_1)}\right]$$

e. To determine high or low point stationing:

$$STA_{\ HIGH\ OR\ LOW\ POINT} = VPT\ STA. - X_T$$

f. To determine high or low point elevation on the vertical curve:

$$ELEV_{HIGH\ OR\ LOW\ POINT} = ELEV.\ VPT - \frac{L_2}{L_1}\left[\frac{LG_2^2}{(G_2 - G_1)200}\right]$$

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