The Commonwealth of Massachusetts

Survey Manual

Massachusetts Highway Department



Metric Edition

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FORWARD

This manual describes survey theory and methods as practiced by the Massachusetts Highway Department and is a guide for all surveyors working on Department projects. A major goal of this Manual update is to introduce the implementation of the metric system of units to Department projects. Although these procedures are recommended as standards in most cases, we recognize that surveyors will deviate from them as conditions warrant. Addenda may be issued and incorporated into the Manual as technology and procedures evolve.

SECTION ONE

GENERAL INFORMATION

1.1 PUBLIC RELATIONS AND THE SURVEYOR'S RESPONSIBILITIES

Surveyors working on Massachusetts Highway Department (MHD) projects are professionals representing a public service agency. Whether a surveyor is an MHD employee or a member of a crew under contract, it is important to the Department that these representatives maintain good public relations in their work.

Surveyors are required to be courteous and tactful with citizens seeking information on a project. The crewchief should be the only spokesperson for the survey party. He/she may answer reasonable questions relating to the project, but should not speculate about possible alignment changes, easements or takings, or any aspect of a proposed design. He/she should not advise property owners or make statements that could cause misunderstandings. Any requests for information or questions of more than a basic nature should be referred to the District Survey Engineer or District Highway Director.

Conduct of survey crew members must be professional and proper at all times. Care should be used when marking survey points. Paint, flagging, and other marking techniques should be used sparingly. Crewchiefs should use good judgment in laying out surveys to minimize property damage and avoid creating hazardous conditions. When line cutting is required, the brush should be removed from the line and disposed of properly.

Where pedestrian traffic is likely, stakes should be hubbed (set flush), or other surface marks should be used. On those occasions when the crewchief must perform calculations in the vehicle, he/she should find duties for the rest of the crew.

1.2 RIGHT TO ENTER PROPERTY

Surveyors should not perform work on private property, or property of any entity other than MHD, without obtaining legal right to do so. District Survey Engineers and crewchiefs should make certain that property owners have given verbal or written permission, or have been notified by the District Highway Director as set forth in MGL, Ch. 81, Sect. 7F, as amended by Ch. 582 of the Acts of 1958 and Ch. 30 of the Acts of 1979:

Entry on private land for purpose of surveys, soundings and drillings

Whenever the department deems it necessary to make surveys, drillings or examinations to obtain information for or to expedite the construction of state highways or other projects under its jurisdiction, the department, its authorized agents or employees may, after due notice by registered or certified mail, enter upon any lands, waters or premises, not including buildings, in the commonwealth for the purpose of making surveys, soundings, drillings and examinations as they may deem necessary or convenient for the purposes of this act, and such entry shall not be deemed a trespass nor shall an entry for such purposes be deemed an entry under any condemnation proceedings which may be then pending. The department shall make reimbursement for any injury or actual damage resulting to such lands, waters and premises caused by any act of its authorized agents or employees and shall so far as possible restore such lands to the same condition as prior to the making of such surveys, soundings, drillings or examinations.

It is standard practice on large projects for the MHD District Office to make a certified mailing to all abutters. This notice should be prepared by District Survey personnel for the signature of the District Highway Director. Property owners generally cooperate, but some may be hostile. An explanation of the need for entry and a request for the owner's permission encourages cooperation. Diplomacy and tact on the part of all survey crew members is required in all cases.

1.3 UNIT OF LENGTH

The standard unit of length for MHD land surveying and mapping is the meter. The 100-foot survey station is replaced by the 100-meter survey station. The standard interval for marking baseline, taking detail and/or cross sectioning shall be 20 meters. MHD conforms to the National Geodetic Survey (NGS) policy of providing state plane coordinates (SPCs) in meters when dealing with the North American Datum of 1983 (NAD 83).

When the previous datum (NAD 27) became law, the "U.S. survey foot" was the standard unit of length. The distinction between the "U.S. Survey Foot" and the "international foot" is a subtle but important one. The "international foot" is defined as 0.3048 m exactly and the U. S. Survey Foot is defined as 1200/3937 m, or 0.30480061 m. The "international foot" is shorter than the "U.S. survey foot" by 2 parts per million. This difference may not be a factor in distance measuring, but can introduce major errors in coordinate conversions if not considered.

By statute and regulation, for conversion of meters to U.S. survey feet "...the meters shall be multiplied by 39.37 and divided by 12 which results in a constant multiplier having a value of 3.280833333333 to 12 significant figures." Surveyors using calculators or computer programs to make unit conversions should be certain to use this factor.

1.4 HORIZONTAL AND VERTICAL DATUMS

The horizontal datum in use by MHD is the North American Datum of 1983, following a 1991 amendment to Chapter 97 of the Mass. General Laws. (See Appendix 1, Excerpts from Mass. General Laws). The ellipsoid that forms the basis for NAD 83 is the Geodetic Reference System of 1980 (GRS 80) Ellipsoid. The previous system, the North American Datum of 1927 (NAD 27), used the Clarke Spheroid of 1866 and was the basis for the State's Lambert Conformal Conic Projection. The difference between the two ellipsoids is the major reason for the coordinate shift from NAD 27 to NAD 83. The GRS 80 Ellipsoid provides improved values for the earth's size and shape and removed previous distortions. Surveys performed for MHD should be in NAD 83 state plane coordinate values.

The vertical datum in use by MHD for many years is the National Geodetic Vertical Datum of 1929 (NGVD 29), formerly known as Mean Sea Level datum. NGS has developed an improved vertical datum, the North American Vertical Datum of 1988 (NAVD 88). MHD is now using the NAVD 88 datum on all recently established bench marks so there will soon be sufficient control to officially adopt the new datum. The Federal Geodetic Control Subcommittee has affirmed:

> ...the NAVD 88 datum as the official civilian vertical datum for surveying and mapping activities in the United States performed or financed by the Federal Government, and to the extent practicable, legally allowable, and feasible, require that all Federal agencies using or producing vertical height information undertake an orderly transition to NAVD 88.

Any questions on local control or the proper datum to be used, or for conversion from one datum to another, should be referred to the MHD Boston Survey Office.

1.5 SAFETY

Survey personnel, whether MHD employees or employees of survey contractors or sub-consultants, are required to conform to MHD safety requirements, applicable safety laws and regulations, and the latest version of the following publications:

Federal Highway Administration's Manual on Uniform Traffic Control Devices (MUTCD).

U.S. Department of Transportation's Traffic Control Devices Handbook, Part VI - Work Zone Traffic Control.

MHD's WORK ZONE SAFETY - Guidelines for Massachusetts Municipalities and Contractors.

U.S. Occupational Safety and Health Administration's Safety and Health Regulations for Construction, Part 1926.

1.5.1 Personal Safety

Survey personnel must wear safety glasses while using any hand tools, including line cutting tools. One of the most frequent tasks crew members do is setting a mark to occupy or to use as a tie. A sledge hammer, the common means of driving stakes, rods or pipes, is also the major source of eye injury for surveyors. Use of safety glasses is extremely important when driving stakes, especially in cold weather when a frost pin is needed to set most marks. Hard hats are also a requirement at all construction sites.

Warm weather attire is also a safety concern. MHD Policy Notice N-90-035 states:

> All employees must report for work suitably dressed for protection against sunburn, various insect bites, and contact with poisonous shrubs/vines. Tube and tank tops, shorts, sneakers, thongs, sandals, or open toe shoes **will not be allowed**. If any employee is not suitably dressed for work, he/she will not be permitted to work until suitably attired.

1.5.2 Roadway Safety

Survey crews spend a good part of their time on roadways of many different types from country roads to city streets to high speed expressways, under a variety of climatic conditions. With electronic instruments becoming more common, the need for survey crew members to be in the traveled way is minimized. However, the crewchief is expected to act responsibly by analyzing every situation and to take any action necessary to ensure the safety of the crew members as well as the motoring public. The roadway shall be properly signed and coned, crew members shall wear proper safety gear and shall stay alert.

SECTION TWO

SURVEY OPERATIONS

2.1 CONTROL SURVEYS

All horizontal data for MHD highway and bridge projects are required to be in the Massachusetts State Plane Coordinate System (SPCS) on the North American Datum of 1983 (NAD 83), except for those excluded by the Department. The SPCS in Massachusetts is a Lambert Conformal (conic) Projection, which is divided into two zones. The Mainland Zone encompasses all counties from Berkshire to Barnstable, the Island Zone is comprised of Dukes and Nantucket Counties.

All vertical data on MHD highway and bridge projects are required to be on the National Geodetic Vertical Datum of 1929 (NGVD 29), or on the North American Vertical Datum of 1988 (NAVD 88).

Some projects, usually small isolated ones, may not be within reasonable distance of geodetic control. The Department reserves the right to exclude such projects from standard control requirements.

The latest available information on the location, suitability, and known condition of Massachusetts Geodetic Survey (MGS) and National Geodetic Survey (NGS) control is available at the MHD Survey Office at 10 Park Plaza, Boston. That office should be consulted before survey work requiring geodetic control is undertaken.

2.1.1 Surveys by Consultants

All surveys performed for the Department by consultants (design engineers, survey contractors, survey sub-consultants, etc.) are subject to the Department's inspection at all stages of a project.

Such consultants are directed to seek advice and information from the MHD Survey Office for the best available control in the area of a project before they estimate survey costs or begin reconnaissance. Control anticipated to be good may be destroyed or inadequate, and new control may have been added that will simplify the survey operation. Circumstances may warrant the Department to expand or readjust control. Survey consultants working on MHD projects will run control only with prior MHD approval and based on MGS or NGS control approved as sufficient.

2.1.2 Horizontal Control

MHD will use Global Positioning System (GPS) procedures to establish control at most MHD projects as a base for all survey work to be performed.

On most large projects, basic horizontal control consists of a random-type traverse connected and adjusted to MGS or NGS control. Such traverses are usually run out using electronic total stations, or theodolites with electronic distance measuring devices (EDMs), of suitable quality to obtain the desired precision.

Any reproduced highway baseline included in the control must be measured and tied in to meet the required precision.

Primary control should be tied in strongly to MGS or NGS control unless other control has been stipulated. Secondary control should be connected to control of equal or better precision.

Unless a closed survey is made, the use of control that is not part of the project's particular net is not recommended since other points may not be compatible.

2.1.3 Horizontal Order of Accuracy

MHD follows the FGCC Standards and Specifications (Appendix A-11) for survey work. If a surveyor deems it necessary to vary from these standards, prior approval should be obtained from the District Survey Engineer or from the MHD Boston Survey Office.

First-Order: applies to control established by static GPS.

Second-Order, Class 1: applies to major bridges, tunnels, and structures of such size and importance on controlled access state highways as to justify geodetic precision.

Second-Order, Class 2: applies to minor bridges and State highways not covered under Second-Order, Class 1, as well as to all other secondary roads except those of minor nature.

Third-Order, Class 1 is a minimum requirement, for minor projects where higher accuracy is not essential.

MAXIMUM ERROR OF CLOSURE

Order of Accuracy:	First-Order	Second-Order Class 1	Second-Order Class 2	Third-Order Class 1
Allowable Error:	1:100,000	1:50,000	1:20,000	1:10,000

Error of closure is only one of the many factors, which determine the accuracy of a survey. Proper procedures and instrumentation as detailed in Appendix A-11, FGCC Standards and Specifications for Geodetic Control Networks, should be used.

2.1.4 Measurement of Angles

Use FGCC specifications, Appendix 11, for angle work. Settings for instrument positions, or sets, should be evenly spaced around the circle according to the formula:

$I = \frac{180}{N}$ where I = Increment
N = Number of sets or positions

With the use of digital reading theodolites and total stations in conjunction with electronic data collectors, the initial setting has become less important. While many data collectors or post processing software "mean" angles automatically, it is good practice to set zero degrees, minutes and seconds on the initial sight in the first set of angles and begin any subsequent set with an increment of degrees. The raw data can then be reviewed to readily differentiate between sets of angles.

In turning angles, considering the direction of progress of the baseline or the traverse, the angle should be turned to the right, with the initial station to the rear.

In closing the horizon with a transit, the initial will be ahead for the second angle. If there are more than two rays, each angle will be turned separately, the last angle closing the horizon.

In using a theodolite/total station, all of the rays may be picked up in each position, the initial being the station to the rear. If there are more than five rays, it may be best to pick up the main rays, using the same initial. Do not reverse the scope on a poor ray. In the case of highly unbalanced rays, use the longest sight as the initial.

If the surveyor finds it necessary to deviate from the rule of turning from behind and to the right, it should be clearly detailed in the notebook.

2.1.5 Measurement of Distances

See Appendix A-11, FGCC specifications, for various classes.

Most distances are now measured electronically. Surveyors should be aware of the limitations of the EDM device and should build sufficient redundancy into their procedures to maintain allowable accuracy. Since a common accuracy requirement for an EDM is +/-5mm + 5ppm, critical distance measurements of less than 30 m will be performed with a standardized (highway) baseline-grade tape.

EDMs and tapes should be regularly checked on one of the five calibration baselines established throughout Massachusetts for the use of the surveying community. (See Addendum A-8, Massachusetts Calibration Baselines).

A) Taping

If measured with a tape, each course should be measured twice, once in the forward and once in the backward direction. Two such measurements must agree within 1:20,000, or they must be repeated until the desired result is secured. Taping should be done with a 30 or 50-meter heavy-duty baseline tape, carefully standardized and equipped with a thermometer and spring balance.

Tapes must be used under the same conditions as when standardized or under such conditions for which a correction can be applied. Sufficient data for determination of corrections for alignment, tension, sag, grade and temperature must be recorded with tape measurements.

1. Alignment

No part of the measured line should be more than .03 m from the straight line between angle points. This aligning should be done with a theodolite.

2. Tension

For tape supported throughout (laid flat), tension should be 4.5 kg. For tape supported on two supports 15 m or more apart, tension should be 9.0 kg. With two supports and less than 15 m apart, tension should be 4.5 kg.

3. Support

Method of support for each tape measurement must be clearly recorded in the notebook. "T-4.5" indicates tape supported throughout, 4.5 kg tension; "2-9" indicates two supports, 9 kg

tension.

4. Grade

Difference of elevation between points of contact of measurements must be recorded in the survey notebook under the heading of "Inclination" or "Elevation Difference" opposite each measurement. Grade corrections are to be calculated or interpolated from grade correction tables, and applied to each measurement.

5. Temperature

Temperature readings should be recorded for each tape length and an average temperature over the section determined. From the temperature correction table, the correction per 30 m can be selected and applied to the entire length of the section.

6. Tape Correction

A correction should be applied to each measurement according to figures derived from standardization.

B) Taping Techniques

Accurate taping can be readily accomplished by measuring with the tape laid flat on a smooth surface such as on railroad tracks or on bituminous concrete. The points may be marked with a pencil on masking tape. Care should be taken to keep the tape straight when marking.

This type of taping is best done in dry, cloudy weather. If the surface follows a vertical curve it will be necessary to determine elevations every 10 m, in addition to the 0 and 30 m points.

Accurate taping may also be accomplished on low taping bucks. Higher stools or tripods may be used with good results where low bucks cannot be utilized.

2.1.6 Methodology for Horizontal Control Surveys

A) Reconnaissance

Control information should be obtained from the MHD Boston Survey Office. Recovered monuments should be inspected for damage or movement, and the physical condition, description, etc., checked. A Recovery Report (see Addendum A-13) should be completed and returned to MHD.

Some requirements for laying out the traverse:

1. Position and azimuth should be connected to known control;
2. Courses should be long and balanced;
3. There should be few sharp angles in traverse alignment;
4. Sight clearance should be about 1 m;
5. Traverse point locations should be sound and practical, with marks not likely to be disturbed or destroyed and well tied down;
6. All main monuments should have azimuth marks. Azimuths may be turned to spires or other prominent features, or azimuths and distances taken to local bounds, e-pins set in boulders, etc., 60 m or more from station.

B) Monumentation

Disks for permanent points should be set in 1.2 m concrete monuments, or in concrete structures, ledge, etc. For semi-permanent points, punch marks should be placed in 48" deformed bars or similar material driven flush with the ground, or countersunk in areas subject to traffic. E-pins should be placed in concrete or stone, spikes in roots or stumps.

Important points should be tied in with three hook ties or two sets of range ties, with survey book sketches and descriptions showing local topo such as buildings, curbs, trees. Station and offset should be used if baseline is available.

C) Field Checks

Standard field procedure is to close an angle traverse by azimuths or bearings, and to check forward and back measurement runs, and all notes and figures. Notebooks must be indexed and cross-indexed, and should always contain a plan or sketch of each project.

The crewchief does not normally adjust the traverse or perform computations unnecessary for field work. However, if a data collector has closure capabilities, it is prudent to check the closure in the field to avoid a return trip. If recording is done manually, enough checking should be done to ensure that the traverse is complete and will close.

2.1.7 Vertical Control

Most existing vertical control is based upon NGVD 29, with new vertical control being established on NAVD 88. MHD may specify NAVD 88 datum for a particular project or purpose, or may allow use of existing local benches, or even an "assumed" bench for small isolated projects.

Vertical control for a project shall be connected and adjusted to MGS or NGS bench marks of an equal or higher order. At least two MGS or NGS benches shall be used and these should be checked for relative difference of elevation.

2.1.8 Vertical Order of Accuracy

See Appendix A-11, FGCC Specifications. Control bench marks should meet First-Order, Class 2 standards. Levels should be run in sections and should seldom be more than 2 km in length. Each section shall be run forward and backward, and the two runnings shall not differ more than 4 mm times the square root of distance, one-way, in km. This work requires the use of one-piece precise (invar) rods and First-Order bench marks.

Second-Order, Class 1 is normally required for major bridges, tunnels and all structures of such size and importance as to justify geodetic precision. Levels shall be run in sections seldom more than 2 km in length. Each section shall be run forward and backward, and the two runnings shall not differ more than 6 mm times the square root of distance, one-way, in km. This work requires the use of one-piece precise (invar) rods and bench marks of equal or higher precision.

Second-Order, Class 2 applies to the majority of MHD projects, most highways and roads except minor ways. Levels shall be run in sections seldom more than 2 km in length. Each section shall be run forward and backward, and the two runnings shall not differ more than 8 mm times the square root of distance, one-way, in km.

Third-Order accuracy is a minimum requirement, to be used where higher accuracy is not needed. Two runnings should not exceed 12 times the square root of the distance one-way, in km.

2.1.9 Methodology for Vertical Control

Although some general guidelines are detailed here, Appendix A-11 should be reviewed for FGCC standards for differing instrumentation and field procedures by order and class.

Level lines shall originate at and close upon First-Order bench marks when available or be run in closed circuits. At least two bench marks, shown by new leveling not to have changed their relative elevations, must be used as the starting point.

Third-Order levels should be run by three wire leveling and be run in sections between permanently marked bench marks not more than 2 km in length.

All lines are to be leveled independently in both the forward and backward directions, preferably under different atmospheric conditions (such as forward in the morning and back in the afternoon).

Portable turning points such as 0.3 - 0.5 m iron pins may be used. Turning plates should be used if pins are inappropriate.

Any permanent or temporary bench mark that is established on the forward run must be included on the back run.

The thermometer should be read to the nearest degree at the beginning and end of each section.

A daily observation of the collimation error of the level should be made and recorded on a separate page of the level book, using the C-Factor method. If "C" is greater than .05 mm/m, the level must be adjusted.

The difference between lengths of foresight and of backsight shall not exceed 10 m.

The length of sight should be great enough so that 5% to 15% rerunning may be expected. Short sights and a small amount of rerunning may indicate lack of production and excessive caution. Longer sights of not exceeding 60 m generally indicate satisfactory production. Experience will show ideal lengths of sight for a survey party.

A temporary bench mark should never be used to begin or end the day's work unless it is properly recorded in the notebook. It must be permanent in nature such as the top of a bolt or pin in a concrete base, a chiseled square on solid concrete, etc.

Setups should be "broken", i.e., the instrument height (HI) at the end of the forward run should be changed before the back run is begun.

Some common leveling errors include:

*Unbalanced backsights and foresights: errors are mostly systematic

(cumulative) due to instrument adjustment error, curvature of the earth and other factors.

* Settlement of the instrument: systematic errors.

* Leveling in windy weather: the instrument and the rod are affected and accurate readings are difficult to obtain.

* Careless instrument work: improper focus (parallax); reading rod with bubble not centered; faulty rod readings due to factors such as obscured rod, sights too long, etc.

* Careless rod work: poor turns; careless rod plumbing; turns not marked; wrong turn used.

* Errors in recording and communications: notekeeper failing to repeat readings for check; failure to write legibly.

While survey crews may not be able to follow geodetic procedures exactly because of practical considerations, they should follow good field practice and strive for precision and accuracy.

2.2 PRELIMINARY SURVEYS

"Preliminary survey" generally means the collection of "as-is" field data for use in design or construction so that accurate project plans, specifications and estimates can be prepared. Needed information includes baseline data, topography, property lines, etc.

The Department now does most preliminary survey electronically using a total station and electronic data collector, often called an electronic fieldbook (EFB). Use of a traditional fieldbook is still required for all MHD survey operations, however, so that supplemental information and sketches can be recorded. EFBs allow substantial time and labor savings in processing and plotting field data to create an "as-built" plan. Much design work is rehabilitation and reconstruction of existing structures so that the as-built information becomes the base plan.

2.2.1 Field Survey and Photogrammetry

On many projects, aerial surveys by photogrammetric methods furnish a major part of the information needed. Traditional field survey work is considerably reduced, but still needed. Field survey is used to establish ground control as a means of measuring and checking the accuracy of photogrammetric mapping, to gather missing data or to provide data of higher precision. For example, field crews will run out location and baseline information, tying into horizontal

control. They will also obtain critical elevations, details of utilities, property lines, drainage, etc. If needed, coverage of areas outside the flight lines or areas obscured by natural features and structures will also be provided.

2.2.2 Preliminary Survey

The Department obtains field survey data for preliminary engineering purposes (whether the design is performed by MHD, or by a design consultant) by using survey parties:

* comprised of MHD personnel,

* provided by survey firms working under contract to MHD,

* provided by a design consultant under the terms of its contract, or by survey firms working as sub-consultants to a design consultant.

Survey work is initiated by written survey request originating with an MHD project manager or an MHD engineer performing in-house design. Survey requests are routed through the office of the Deputy Chief Engineer for Highway Engineering and the Boston Survey Office to the appropriate District.

Before authorizing the start of survey work, the District Survey Engineer should obtain a copy of the applicable Engineering Work Order (ADM 720), which provides the activity code, job number, expenditure accounts and other project data.

The District Survey Engineer will furnish survey parties with instructions and data needed such as plans, survey books, administrative information, etc. As the survey progresses, the crewchief may acquire additional materials such as property plans, deeds, record layouts, utility information, etc.

MHD survey notebooks are required to be used for all surveys performed for the Department whether or not data is gathered electronically. Horizontal data (detail, baseline and traverse information) is kept in even-numbered notebooks, with odd-numbered notebooks used for vertical data (levels, profiles and cross sections).

The District Survey Engineer will supervise and periodically check on the conduct of the work.

Preliminary Survey will include the following operations, as needed:

* Horizontal control should be recovered (i.e., located in the field) and extended as needed.

* Existing baselines and other important lines should be run out and connected. This work should be checked in the field with subsequent review of computations and adjustment. Final or proposed final location lines should be run out and connected. All important points should be tied in.

* Existing vertical control should be recovered and bench runs made, checked, computed, and adjusted.

Detail and cross sections should be taken from proper baselines. On some projects, this data may have to be located from random or supplementary lines. EFBs allow most of this data to be obtained from random traverse lines, which reduce roadway traffic hazard as well as transcribing errors.

Property corners should be recovered and tied into survey lines. Angles and distances to physically evidenced, or approximated, property sidelines should be observed and recorded.

Bridge and stream surveys are made as directed. (See Section 2.2.8, Bridge Surveys).

Other work, under general classifications of cross sections and detail, includes punchings, soundings, ledge sections, grid sections, drainage, location of utilities, etc. (See Section 2.2.7, Cross Sections and Related Work).

In addition to running out final location baselines, the survey crew may make surveys for right-of-way locations, ramp locations, structures, etc., including some work which is not strictly "as is" but which will prove useful for design or other purpose of the work.

Appendix A-3, Abbreviations, contains MHD field work and design standards for illustrations, abbreviations and plan symbols. The MHD Highway Design Manual also provides much useful information.

2.2.3 Base Plans

A base plan, also known as an "existing conditions" plan, is a graphical representation of the preliminary "as is" survey data. MHD requirements are detailed here to help the survey crewchief understand what field data is needed for base plan preparation. While field personnel will not usually be involved in the actual plotting of the data, they should fully understand how field data is

utilized in the preparation of an acceptable base plan.

A) Survey Field Notes

When the field survey work is completed, whether performed manually or with EFB, the notebooks containing alignment data, details, sketches, and bench level and cross section notes should be checked for completeness. For major projects, survey baselines and traverse lines are computed and adjusted by the MHD Boston Survey Office. The surveyor shall use MHD methods to adjust all other baselines, traverses and levels, to the closure limits in Appendix A-11, FGCC Standards and Specifications for Geodetic Control Networks, or as directed by the District Survey Engineer. Any discrepancies in field notes shall be carefully checked and remeasured, if necessary. Adjusted data should be checked in the field.

B) Data Obtained Manually

The base plan should show all cultural and natural details within the area of the proposed project. In general, base plans are to show all structures with the type of each noted, as well as water lines, sewer lines, drains, underground utilities, poles, manholes, hydrants, catch basins, wells, curbing, roadway edges, street signs, streams, bridges, etc. The plans shall show the location of state, county and town layouts, approximate property lines, owners and deed and Land Court numbers.

C) Data obtained by Electronic Fieldbook (EFB)

A base plan by EFB is a two-dimensional (2-D) (planimetric) drawing of the existing detail within the project limits. This is most commonly generated from three dimensional (3-D) electronic data collection, but is prepared as 2-D lines for ease of editing and adding design data.

The plan should contain plan symbols and line types in conformance with MHD Highway Engineering Division drawing standards. Sample abbreviations can be found in Appendix A-3. The Highway Design Manual shows a sample base plan and details additional requirements.

A survey reference line, established in the field by random traverse between geodetic control points, shall be shown on the plan. When required, a supplemental center line can be created by splitting some suitable feature (bridge structure, roadway surface, etc.).

A Digital Terrain Model (DTM) will be compiled from the 3-D field data and checked for correct location of break lines. Contouring of the DTM will be done at an interval between 0.2 and 0.5 m, with the concurrence of the District Survey Engineer. The DTM (with contours) will be submitted in electronic format along with the base plan.

The final submittal to MHD should include all relevant electronic files that the CAD software may create.

2.2.4 Baselines, Sidelines and Property Location

The term "baseline" has acquired a variety of meanings over the years. Its definition herein is: a reference line of known dimension and bearing, from which various data can be related through field measurements.

A) Some common baseline types are:

1. Survey Baseline - Until electronic data collection, the Survey baseline has historically been the reference line laid out on the ground by the survey party so that detail and cross sections could be tied by station and offset and recorded in a survey notebook. For a new roadway in virgin terrain, the survey baseline would often be a random traverse line located in the field by the survey party. In some cases a preliminary alignment would be given to the survey party to lay out. On an existing roadway this line would be established from monumentation and appear similar to the baseline of record. It would be an approximation, however, because distances would be apportioned both longitudinally and transversely to conform to record stationing. Electronic data collection allows the use of a random traverse line with angle points situated off of the traveled way so that the same data can be obtained with less safety risk and less traffic disruption.

2. Record Baseline - The baseline of record, or layout baseline, is the original line laid out and on which the Layout Plan, and the order of taking (or county decree) are based. It is rarely possible to exactly recreate the baseline of record because the original ties are usually destroyed. When this line is required, it has become standard practice to recreate the line from the best witnesses available: the sideline monuments that were set from the record baseline during construction.

3. Construction Baseline - The baseline provided on the construction drawings, also called the design baseline. This line is most often, but not always, the centerline of

construction. It is created by the designer to make estimating, design and construction operations easier. Construction plans should include detailed sketches of ties relating the construction baseline to the survey baseline at several locations, by geometry and by coordinates.

4. Historical Baseline - any documented line that can be recreated from said documentation. This category could include any of the baselines described above.

B) Baseline Layout

The first step in making the preliminary survey is to establish state plane coordinates at the site using geodetic control. Next, lay out and set a survey baseline (random traverse) and then the control points of a historical baseline if records exist: the PCs, PTs, PIs if accessible, and the angle points. The survey baseline is an important reference line on the project, since location, survey and right-of-way data is referenced to it. Since the baseline is a prime control line for the project, it must be accurately located. Standard baseline layout stationing shall be at 100 m intervals. If the survey is to be by EFB, it is recommended that the random traverse line be established to accuracy equal to a manual survey baseline. Baseline control points shall be located with particular care. Specifications for measuring may be found in Section 2.1, Control Surveys.

On major MHD projects, predetermined alignment data is furnished to the District Survey Office during the design phase. This information should be provided on plans and other documents on which the proposed alignment, existing control and connections, coordinates, angles, and distances are shown.

The survey crew should run out the preliminary baseline promptly to determine how well the lines will fit on the ground. Record data will normally be held despite minor discrepancies. Errors, large or small, should be called to the attention of the District Survey Engineer. Baseline and layout data should be adjusted before the layouts are recorded.

Some preliminary survey requests are for EFB surveys on reconstruction projects for which no record baseline information is available. A "survey baseline" may then be created by splitting some permanent feature, e.g. bridge roadway, stone wall, etc., and incorporating it into the traverse. The random traverse line is then the most important line for the project as all information will be referenced from it and any property or layout monumentation should be tied to it.

When there is an option, stationing for main baselines should run

south to north, and west to east.

Baselines and traverse points must be well tied down with strong semi-permanent markers such as range ties extended beyond any contemplated construction. The baseline should extend 200-400 m beyond the limits of the project, or as needed for the preservation of such points.

Traverse line and baseline measurements and notes should be recorded in even numbered MHD notebooks (HED 750).

C) Reestablishing Baselines and Sidelines

Replacing baselines and sidelines is a common survey function and may range in difficulty from recovering a section of sideline to within 0.3 m to rerunning the baseline and recovering the sidelines of a major highway for alteration or reconstruction purposes.

For recent roadway layouts, record data such as layout plans and notebooks are usually readily available, making the survey a relatively routine process. However, the work is of some technical difficulty, especially on limited access highways, requiring survey personnel with skill in baseline work. The party must check all work possible, add ties where needed, and note any discrepancies.

In working on older roadway layouts, surveyors will generally run into problems. Bounds may be missing, damaged or out of position. The crewchief is responsible for accurately locating the original sidelines even if incorrectly set originally. The surveyor must also recover all physical evidence, including Massachusetts Highway Bounds (MHBs) and any other indications of ownership, such as property corners, fences, walls, etc. This information is a necessary part of both detail and baseline notes. The surveyor should also recreate the baseline, including a complete rerun for readjustment, if needed.

Small rural roads which have not been defined either by plan or by written description can usually be located by physical evidence, the most common being the location of stone walls which parallel the traveled way. Measurements should be taken between the faces of the walls, considering the walls themselves as belonging to the abutters. Often the approximate distance between them will be a commonly used number of rods (1 rod = 16.5 ft. or 5.029 m), such as 2, 2 1/2, 3, etc., although the distances can vary considerably. It should be noted that the records of many legally defined roadways have been lost. It is important that the surveyor locate all physical data such as fences, property corners and buildings, and should be

particularly alert to evidence of metes and bounds descriptions such as inscribed granite markers. All of this data should be shown as detail.

Originally, many rural roads were simply rights-of-way, with abutters' properties meeting within the way and not necessarily in the middle. Such ways may later have been defined by town meeting action, but often were not recorded at the County Registry of Deeds. A Massachusetts Land Court document prepared for surveys performed under its jurisdiction, Manual of Instructions (1989), may be followed as a means of locating these roads. Section K, Streets and Ways, defines "way" as follows:

The word way as hereinafter used will include all highways, boulevards, avenues, roads, streets, paper streets, traveled ways, cart paths etc., whether public or private, constructed or not, in use or not, existing physically on the ground or legally of record.

Survey notes should indicate whether a way is public or private.

Monuments such as stone bounds "supposedly located at the termini of the curves and at the angle point of the ways.... must be surveyed as they exist on the ground." They should be identified by type (e.g., C.B., MHB, S.B.), mark (e.g., d.h., e.pin), and if "held", be so noted. (See Appendix A-6, Sample Survey Notes).

D) Property Location

Property surveys performed for the Department vary in scope from the taking of routine detail to a determination of complete property boundaries. Although thoroughness of property work should be limited to only what is needed, the surveyor may also be required to obtain additional data and establish new property lines, etc. Accurate boundary locations are required for takings and easements. All available evidence should be researched and weighed according to land surveying principles; the locus should be determined from the best evidence, and the property tied in accurately to the baseline or traverse line. All research data should be turned in with the survey. Surveyors are expected to understand and apply principles of property survey so that they provide reliable data.

Property surveys involving registered land, or land proposed to be registered, must be performed according to the Massachusetts Land Court's Manual of Instructions, available at its Boston office and at the Registry of Deeds in the various counties.

E) Specific Instructions and Order of Calls

The District Survey Engineer shall specify in detail to the crewchief any property work needed for MHD survey purposes. When a land taking is involved, a plan of the entire property and buildings may be required. Although it may not be necessary to run an entire field survey, the proposed taking and connections to the remainder must be measured. Plans of proposed private projects that abut a roadway like subdivisions, plazas, industrial parks, etc. should be obtained and tied into the baseline.

For a taking involving registered land, a complete resurvey need not be made. The Land Court requires that a subdivision plan be prepared whenever a portion of a registered parcel is taken. Therefore, it is MHD policy that only easements will be taken on registered land unless an entire parcel is taken, as it exists.

The corners and lines of the affected areas should be located in relation to the baseline and any Land Court monuments found should be located to aid in locating the parcel in relation to the highway layout. Angles should be turned connecting the remainder to the taking. The party should recheck any discrepancies found in old angles and distances. Bounds that are recovered should be noted as "fnd," with physical condition noted, and "quick-tied." Conflicting evidence and descriptions are commonly encountered in this type of work. The surveyor should locate the property from the evidence, not relocate it. The "order of calls" should be considered, in the following order of importance:

1. Natural monuments
2. Artificial monuments
3. Adjoiners' or abutters' lines
4. Courses and distances
5. Area

Layout evidence such as property corners and courses should be shown in the detail for every project whether or not it appears significant. Fences, walls, etc., along the layout should be located to 0.05 m. Property lines may be ranges from the baseline with two sets of angles turned to them, and measurements made to corners.

F) State and Town Lines and Corners

Town lines run from angle point to angle point (bound to bound). Road stones are often found on line between these bounds at all intersections with highways, in accordance with MGL Chapter 29, Section 4, as amended. Road stones do not carry the legal weight of town corners and should not be considered correct unless confirmed by survey measurements. Many road stones have been shown to be in error by 50 m or more. Road stones placed by MHD are surveyed from town corner to town corner or located by coordinates, with the work documented in survey notebooks.

Surveyors performing preliminary location surveys should locate town corners and road stones in the area.

G) Town Boundary Atlases

Town and city boundary lines, which sometimes are also county and state boundary lines, are thoroughly described and illustrated in the Town Boundary Atlas series, published by the Harbor and Land Commission, circa 1900. Copies of the Atlas may be found in city, town and county offices, although the surveyor is cautioned to check for any possible subsequent revision by the Legislature.

The MHD Boston Survey Office keeps an up-to-date Atlas set available for reference and can also provide approximate state plane coordinates (+/- 0.3 m) for town corners that have not been surveyed for many years.

Any surveyor who recovers a town corner should make recovery notes of the location and condition on HED 697, REPORT ON CONDITION OF SURVEY MARKER (See Appendix A-13), and should redescribe and tie these valuable monuments when necessary.

2.2.5 Detail

A) General Procedures

Detail may be called the "plan view" or top view of a project, topography taken on a horizontal plane, while contours and other vertical information are considered topography. Now that detail is obtained electronically, vertical data is captured at the same time.

The collection of detail generally begins after the baseline or traverse line is set. Detail includes all natural and cultural features on, above, and below the ground in the proximity of the baseline, and those more distant from the baseline, which may affect proper design of the project. The survey crew must be

alert to picking up all necessary detail, including objects not specifically detailed herein.

Appendix A-6 shows typical examples of detail notes. Generally, the surveyor should follow the examples but should make changes to suit conditions. The main purpose is to take easily understandable notes conducive to recreating the survey, should it be necessary.

Traditionally, detail has been taken by station and offset (or by angle and distance) from the baseline. Features not within convenient distance from the baseline may be taken from auxiliary loops closing on the baseline. Detail such as the outline of swamp or wooded areas, meandering streams, ledge outcrops, etc., is often measured by stadia.

B) Procedures for Collecting Detail Manually

Two cloth tapes should be used, one for baseline and the other for offsets.

Stations should be made visible from a distance by use of tags, keel, etc.

A square-off (right-angle prism) should be used at stations and other points on the baseline where precision is needed.

Offsets should be taken at right angles to the baseline at whole stations, at 20 m intervals, or at 10 m intervals if required. For a baseline on a curve, detail should be taken at a right angle to the chord of a 20 meter arc interval. Between station intervals, normal is 90° to the tangent if the baseline is on a tangent, or if on a curve 90° to the chord of a 20 m arc, with measurements made to the chord.

Stationing for detail should be to the nearest 0.1 m, with offsets to the nearest 0.02 m. When using an instrument, stationing should be to the nearest 0.02 m. Offsets to property corners and other monuments should be taken by steel tape to 0.005 m. Other offsets such as wood lines, edges of fields and detail fairly distant, etc., may be taken by tape or stadia to 0.3 m.

A symbol such as a small arrowhead should be used in the notebook to show the distance measured to an object, as in measuring to center of a manhole, middle of face of curb or curb inlet, pole in concrete base, etc.

Detail features should be clearly but briefly identified, e.g., "balanced stone wall", "NYNEX #31", "0.15 m sq. SB up 0.3 m."

Standard MHD symbols and terminology should be used. Refer to Appendix A-3, Abbreviations, and MHD Highway Design Manual.

Notebook notes and sketches should not be crowded. A straight-edge or "fish" (whale) should be used. Sketches, numerals and words should be clear and neat. Detail can be stretched over double pages if needed for clarity, with cross-references for derivation of baseline, layout used, notebook number for cross sections, etc.

C) Procedures for Collecting Detail Electronically

The features to be located are virtually the same as in manual detail collection, but the surveyor must be aware that three dimensions are obtained with every shot.

The crewchief must ensure that the rodperson calls shots correctly. There are many cases where the elevation obtained will have no bearing on the DTMs, and must be noted in the EFB or notebook. Examples are shots of overhanging objects, top of hydrant, top of wall if not on level terrain, and any shot were the prism pole is not held vertical.

Feature coding (attributes), also referred to as string labels or point codes, must conform to the latest MHD feature code list. Since the list is being continually updated, it is not included herein. The latest list should be obtained from the MHD Boston Survey Office when needed.

Due to the convenience and speed of electronic data collection, many surveyors feel sketching detail is less important for EFB surveys. A notebook is required, however, and is to include sketches of the roadway and any major structures within the project limits. Most building dimensions and other data and comments which cannot be recorded in an EFB should be noted in a sketch in the same even numbered book as the traverse sketch. While the sketches will not require the minute dimensions and detail necessary for a manual survey, they should be explicit enough so that an office engineer can orient position on the electronic drawing and complete the base plan. Occasionally writing a point number on the sketch of a prominent detail shot is an aid to the office person working with the data. The importance of detail sketches is emphasized because often the designer working with electronic data does not have ready access to the surveyor who collected the data.

D) Features to be Located

Detail feature locations to be recorded at whole baseline stations and at 20 m interval stations shall include all routine topography that runs along the route such as edge of pavement, medians, traffic islands, curbing or berm, sidewalks, fences, walls, brooks, ditches, edges of swamps or lakes, tree and brush lines, etc. Objects normal to the line of the station, such as points on islands, drives, buildings, etc., should also be collected.

Detail to be picked up at odd stations (in between whole stations or interval stations) should include any changes or additions that occur to the running features mentioned above, such as the beginning or end of a feature, change of type, change of direction, or ending of walls, fences, walks, wood lines, etc. In order to correctly plot the detail in between stations, additional measurements for running features should also be collected, as several additional points are needed to plot curved or irregularly shaped pavement, curb, walks, drives, fences, etc.

Features must be described using brief, clear engineering language, e.g.: Type I, paved w.w., sloped edging, 12-inch R.C., brick retaining wall. If uncertain, use "?".

1. Buildings

Locate, measure and describe all buildings as needed for the particular project. Generally all buildings in the proximity of the baseline must be located for all construction projects.

Buildings within 30 m of the baseline may be located by station and offset to corners by range ties and angles. A combination of ties from different stations may also be used. At least one extra measurement should be obtained for a check.

A building may also be measured by angle and distance to more than two corners from one or more stations. This method is advantageous for locating buildings some distance away, and is most easily obtained by EDM.

The entire building including garages, entrances, walks, walls, etc., should be located so that measurements can be checked against one another. Steps should have the number of risers and treads noted.

The street number of building, owner (if known), type, construction, and use should be identified. Facts which may affect design such as "trailer truck terminal", or "car wash--busy" should be noted.

2. Trees, Shrubs and other Growth

Shade trees, ornamental trees, shrubs, hedges, etc., inside the proposed layout should be located and measured. Orchards and crop areas in or near the layout should be located.

Locate and measure individual trees where practicable. In wooded areas show wood line, describe types of trees and sizes: e.g., "hardwoods, 0.1 m to 0.3 m dia." or "pine grove, pines 0.2 m to 0.4 m dia."

Measure single trunk trees one meter above ground level. Measure multiple-trunk trees (separating below the one meter level) where the multiple growth begins.

In taking detail, the crewchief should keep in mind that MHD Standard Specifications contain a clearing and grubbing item for large areas, paid for by the hectare (2.471 acres), for removal of brush and trees up to medium size. There are separate payment items for removal of individual trees: from 0.25 m to 0.6 m in diameter, and over 0.6 m in diameter.

3. Public Utilities

All utility features should be accurately located and recorded in the notebook, with sketches as needed. If utilities or related data are on different pages of the notebook from the regular detail, the respective pages should be cross-referenced.

Telephone and electric towers, poles and other structures in or near the layout area must be located. It is necessary to know the name of the utility, and the direction and height of cable above the ground. The cable height should be measured by horizontal distance and vertical angle, but contact with cables or wires should be carefully avoided.

Other utility detail features such as poles, hydrants, manholes, gates, etc. should be located and identified by type, company name, number, etc. Storm drains, water, sewer and gas lines, electrical and telephone conduits and other underground utilities should all be located, with the size, type and depth below surface noted.

It may be necessary to obtain help from utility companies to get all pertinent data. If utility companies are reluctant or uncooperative in marking their equipment, lines, etc., the surveyor should locate whatever he/she can and note in the book: "Approximate locations only. Contact the utility company

prior to construction."

4. Private Utilities

All wells in the vicinity of the layout should be located. The height of water from the top of well should be determined.

Private sewage disposal facilities should also be located and identified (e.g., cesspools, septic tanks or disposal fields).

5. Drainage

All drainage features found along an existing highway or road should be accurately located and identified in the survey notebook. All structures and pipes should be dimensioned. For the sake of clarity, it is often advantageous to show drainage separately from other detail, using separate sketches with cross references to regular detail notes.

Where a culvert crosses the baseline, locate the angle of intersection and determine the direction and distance to structures such as headwalls. Locate drainage and streams out to a distance of 30 m.

Drainage sketches should show not only structures such as endwalls, headwalls, inlets, manholes, but also lengths, types and sizes of pipes, location of ditches, etc.

6. Railroads

Where a railroad crosses or parallels the baseline, complete detail should be taken including type and condition of crossing, length and width, protective devices, location of rails, ditches, drainage, communication poles, etc.

It is often necessary to take detail and cross sections for some distance along the railroad baseline or random line parallel to the tracks. On important projects, the railroad baseline will be run out and connected to the survey baseline.

To eliminate the need for minus stationing, set the intersection of the roadway baseline (or other random line) at Sta. 10+00.

2.2.6 Highway Levels

As detailed in Section 1.4, Horizontal and Vertical Datums, until MHD officially adopts NAVD 88 as the vertical datum, the Department will designate either NGVD 29 or NAVD 88 as the vertical datum for use on a particular project or operation. The MHD Boston Survey Office can furnish copies of topo sheets upon which up-to-date control is plotted, as well as description cards for bench marks and other control. Bench marks are shown as squares; traverse stations, shown as triangles, often serve as bench marks also.

Two bench marks of known elevation should be recovered or placed at the beginning and also at the end of the leveling project. These should be First or Second-Order BMs. The crewchief should run check levels between adjacent benches. If other bench marks are found on the project or adjacent to it, they should be included in the level project. The surveyor is cautioned against using stray bench marks for level work on a project without first physically tying them into others to find if they are compatible. The crewchief should make a recovery report on every BM found, or searched for and not found.

Highway bench marks consist of a series of permanent and semi-permanent points which are identified by consecutive numbers or by numbers and letters. The points should be set where they will be accessible and not likely to be destroyed during construction. They should be carefully described in the notebook by type of mark, general description and station and offset. Heavy spikes (including railroad spikes) are often used as bench marks. They may be driven vertically into tree roots or horizontally into poles or trees so that they cannot be driven further in or off elevation.

Monel steel or brass plugs may be set firmly in concrete or stone. Square cuts in concrete or stone make excellent marks and are not easily confused with horizontal control. Care should be taken not to damage private property. Use of unsightly witnesses such as paint, stakes nailed to trees, etc. should be avoided.

Benches should be established at 300 m intervals or at elevation differences greater than 15 m. Structures require two adjacent bench marks; structures over water should have two benches on each shore. For surveys in urban areas, benches should be placed one or two blocks apart.

FGCC specifications for Third-Order levels are suitable for most highway work. (See Appendix A-12).

All bench marks should be actual turns and not side shots. At an end-of-run bench mark, the instrumentperson should "break" the set-up before a new backsight. Forward and back runs must be taken; single runs are insufficient.

Upon completion of the project leveling, the crewchief shall adjust the leveling to fixed benches first, and then to individual loops.

The single wire method of leveling may only be employed for the least precise work. Three-wire readings are preferable. Sample level notes are in Appendix A-6.

Equipment used in project leveling should consist of a good quality compensating-type level, rod levels and two Philadelphia or California (Frisco) rods adjusted for length. Backsight and foresight lengths should be kept about equal. The instrument person can balance sights by checking the lengths by stadia, or by pacing. Readings should be estimated to half the smallest division in most cases.

The circular bubble on compensating levels should always be kept in adjustment. A peg test should be performed once a week to ensure that the "C" factor does not exceed 7 mm per 100 m.

A compensator may stick, so the level should be lightly tapped before taking a reading. The operator should not re-level the instrument since the instrument height would change. The operator should be wary of the possibility of settlement, since the compensating level gives no indication of movement.

2.2.7 Cross Sections and Related Work

Cross sections show the vertical, or "elevation" view, of a project as compared with detail, the horizontal or "plan" view.

3-dimensional EFB-type data collection automatically produces suitable "preliminary survey" elevations provided:

* The traverse points occupied have been leveled,
* Care is taken by the operator in pointing on the prism, and
* Accurate heights of instruments (HIs), targets (HTs) or rods (HRs) are recorded and entered into the EFB.

Cross sections are normal to a baseline at regular intervals, usually 20 m or 10 m stations. Cross section data may also include odd sections, profiles, drainage and utility elevations, building sills and floor elevations, and other pertinent vertical information.

Related vertical work may include grid sections, ledge sections, different types of profiles, as well as punchings, soundings, boring layouts, etc.

Cross sections plotted from photogrammetry may be acceptable for design as preliminaries, depending on the accuracy of the mapping and save considerable field work expense. Some field work may be necessary to supplement the photogrammetry, such as for ground not visible from the air, when obscured by trees, brush, grass, water, etc., or when additional information is needed.

A) Preliminary Cross Sections

Preliminary cross sections are primarily for design, but are quite useful during construction as the basis for calculation of quantities and for reference purposes.

Where there are abrupt changes in terrain, cross sections should be taken both left and right even if the abrupt change is only on one side. Extra shots should be taken when the baseline profile changes markedly.

Features such as type of pavement, curbing, catch basins, utility structures, walls, etc., shall be clearly identified. Stations, offsets and descriptions should be consistent with the detail notes.

Typical cross section features to be identified are:

1. Crown and edges of pavement, breakdown lane, shoulder, curb, sidewalk, with readings to 0.02 m horizontally, 0.01 m vertically.

2. Walls, such as "balanced field stone", "stone masonry", properly described, showing height, thickness, etc.

3. Standard interval cross sections, noting character of ground, as "lawn", "cultivated", etc. Readings to 0.1m horizontally, 0.05 m vertically at 10 m spacings and at slope breaks, to beyond end of estimated construction limits.

4. Exposed ledge, size and location. Grid coverage may be required in large areas. During construction, ledge may be stripped and more thorough grid section coverage taken.

5. Plus and minus sections where a wall or some feature causes significant vertical change. (Slightly different offsets for the two should be used.)

6. Center of drives, dirt roads, paths, etc., profiles starting from baseline, with amount of skew, if any, noted; floors of garages or other buildings, top of loading platforms, etc.

7. Road drainage systems, showing drainage layout, manhole and

other structure elevations, inverts, sumps, and type of pipes.

8. Ground and sill elevations of buildings, or junction of siding and foundation; other pertinent elevations on buildings such as ground floor of warehouses, service centers, loading platforms, heights of openings, etc.

9. Top and bottom of steps, number of risers and treads.

10. Ground elevations of trees, which may be impacted.

11. Elevations at top of well casings, recording type, water elevation and depth, if possible.

12. Private sewage disposal components, such as cesspools, septic tanks, leaching fields.

13. Adjacent bridges and culverts, (see Bridge Surveys) showing flow lines, top of opening, roadway, water level, reporting on flood levels.

14. Culvert systems (see Section 2.2.8, Bridge Surveys). Obtain skew angle at baseline if system crosses the road, showing complete profile. A complete sketch of structures, with elevations, showing size and type of components should be taken. Brooks, streams or ditches should be located, with profile of bed, top of bank and cross sectional dimensions shown. The channel should be followed for a sufficient distance, say 50 m, and a spur line run, if necessary. Present water elevations should be obtained and any previous high water marks recorded.

15. Intersecting or adjacent roads, streets or railroad track. Cross sectioning should be done on an auxiliary baseline. At intersecting streets, gutterline and top of curbing should be taken for at least 30 m, if needed for pavement design.

16. All underground utilities that can be accessed. Elevations should be recorded and additional information obtained from utility companies, if available, on water gates, gas gates, manholes, gasoline station features, etc.

17. Overhead utilities such as power lines. Height should be taken by vertical angle only, not by throwing any kind of tape over the wires or by measuring with any type of rod, even fiberglass. No person or piece of equipment should be allowed within 4 m of overhead wiring.

B) Cross Section Methodology

Odd numbered MHD notebooks (HED 721) should be used for all vertical notes: cross sections, drainage inverts, soundings, boring notes, as well as for bench level runs.

At the start of cross sectioning, make relevant notes in index and on first page being used: town, road, job number, kind of work, activity, stations, crewchief and other survey personnel, date, page, etc. Note cross reference information such as layout name or number, baseline notebook, detail notebook, etc.

Take cross section notes from top of page down, running right hand sections increasing to right and left hand sections increasing to the left, as shown in Appendix A-6, Sample Survey Notes. Do not crowd notes and leave room for checking and calculations. Leave at least three spaces vertically between sections.

Dimensions at a point on the cross sections are shown as numerator and denominator, numerator being the offset, and denominator the rod reading.

When backsighting to a bench mark, record elevation and a brief description including station, offsets and source. A bench mark which is held should have its elevation shown in parentheses. Place HI value over a crow's foot so that it is clear which cross sections are taken from that particular HI. Clearly describe turning points so that they can be found and used again.

The instrument should be set firmly into the ground as it may be susceptible to settlement if located at a point for an extended period of time. At the end of the setup, the backsight value should be checked before making the foresight on a turning point (TP). Backsight and foresight lengths should be kept as near equal as practical.

Closure into a bench mark should be within about 0.01 m. If good closure is not affected, a new run through the TPs should be made and shown in the book. Wrong elevations can be corrected in the cross section notes by drawing a line through the old elevation and writing the correction above. All adjustments should be explained and referenced.

Features should be adequately described in cross section notes, using abbreviations where necessary. (See Appendix A-3, Abbreviations).

If a section or profile is not normal to the baseline, write "ask" for askew and show angle.

Encircle shots, which have no bearing on quantity, such as sills of houses, manhole rims, etc.

Some data such as drainage studies may be shown on separate pages for clarity, with appropriate cross reference made.

The crewchief should have other party members check the notes for such things as missing sections or elevations, calculations, etc.

C) Punchings

Punching rods are lengths of bars or pipes, which may be extended in length by coupling additional sections as the rod is pushed into the ground. Punchings are taken in soft materials like peat or muck. The surveyor pushes the rod into the peat, muck or similar material until the point of the rod hits hard ground and meets "refusal". Measurement is taken of the depth of penetration and recorded.

Cross sectioning is done first, with punchings taken on the cross section lines.

D) Borings

It is often necessary to lay out boring points for design. The surveyor locates the boring, records the boring designation and elevation of the ground at the point. If the point is on pavement, paint marks are used.

Bridge borings must be located to within 0.3 m of their specified position. If they cannot be located within said tolerance, the designer should be contacted. If they cannot be initially laid out by station and offset, they should be later redefined in reference to the baseline.

E) Soundings (see Section 2.2.8, Bridge Surveys)

Most soundings are taken in lakes and small rivers and streams, and occasionally in tidal or larger bodies of water.

Generally speaking, a baseline must be laid out on one or both shores so that the sounding can be located horizontally. On a small stream a range can be set up so that a boat can be kept on line. Range lines of soundings should be as near 90° to the thread of the stream as possible. A tape may be laid out, and distances measured directly, or a rope or wire may be extended across the stream and tagged at intervals. The baseline on the shore may be a random traverse line setup so that the soundings will be close to 90° from the center of the stream. If the stream

follows a sharp curve, the soundings may be taken radially from a single point from PC to PT of the curve, with the angles of the radial lines recorded. Another method is to send the boat down a range line and have the instrumentperson locate the distance by stadia from the baseline.

For winter work when the ice is thick enough, small holes may be chopped through the ice as people do when ice fishing, so that soundings can be taken.

A water level gage is often needed near tidal-influenced water crossings. The gage should be tied in to two project bench marks. If the level of the water varies during the day, frequent reference to the gage should be made and recorded in the level notebook with dates and times of readings.

2.2.8 Bridge Surveys

Bridge surveys are basically three types: over roadway, over water, and over railway.

In the case of bridges over water, studies are made by the MHD Hydraulics Section, the Bridge Section and the Highway Design Section based on the data gathered in the bridge survey. A complete survey must be made of the stream, dam, or any other controlling features of the stream that would affect the passage of water under the bridge.

All the survey notes pertaining to the bridge and stream are to be kept on separate pages in the notebook and so indexed in the front of the book.

A) Bridge Detail

If there is any possibility that the existing bridge will be used in the construction of a new bridge, special care should be taken to accurately locate and measure the structure. If it is evident that the old bridge is in such condition or location that it has no possible use, the measurements may be taken with less care.

On the substructure, the angles of the abutments with the baseline, the location of tops and bottoms of batters, the widths of bridge seats and parapets, the location of the angles of wings and abutments, the length of wings and widths of copings should all be measured and the foundation located if possible. The type of masonry in the substructure and its condition should be noted.

The extent of detail to be taken on the superstructure is largely a matter of judgment. In all cases the general layout, such as

curbs, trusses, girders, fences, sidewalks, copings, ends of bridge, etc., should be located. If the bridge is of an old truss, girder, or timber type, it will probably not be used. However, if it is of the beam and slab type and in good condition, it may be used and should be carefully measured.

While the highway plans are plotted to the scale of 1:500, the bridge plans are plotted to either 1:250 or 1:100, so that the accuracy on bridge detail locations should be far greater than on general highway work.

Levels should be taken on all parts of the substructure and superstructure, such as the bridge seats, top and ends of wings, bottom of beams, gutters, top of curbs at intermediate points and ends of curbs, tops of slabs, and on footings, if possible. All levels should be referred to NAVD 88. (See Section 1.4, Horizontal and Vertical Datums).

B) Bridge Grid

The bridge grid is taken in order that an accurate calculation can be made of excavation quantities for the proposed structure. In general, shots should be taken on a 3 m grid with additional shots as necessary for abrupt changes in contour. If the ground is fairly uniform, the shots need not be as frequent. They should extend about 20 m either side of the baseline and should cover enough ground longitudinally for any size or type of structure. The grid should be carried under the existing structure.

Wherever a railroad is crossed, the railroad baseline should be picked up and sections taken every 20 m perpendicular to the tracks for a distance of about 80 m either side of the location. These sections should cover the rails, ditches, slopes, and adjacent ground.

C) Stream

If the crewchief believes there is a possibility of a channel relocation, enough of the existing stream should be located to make a complete study of this change. In any event, the stream banks should be located for a distance up and downstream of at least 150 m. Any tributary entering the stream near the bridge site, either above or below, should be located for a distance of at least 150 m from its junction.

In addition to the bridge grid, cross sections perpendicular to the stream baseline and extending out beyond any known flood height should be taken every 20 m for at least 80 m, both up and downstream. Beyond 80 m, a profile should be taken to definitely establish the grade of the stream's bed.

For any down-stream tributary mentioned above, cross sections for 80 m should also be taken.

1. Dams

Any dam immediately above and any below the bridge site which in any way affects flow through the bridge should be measured for the calculation of the flow. A section across the dam spillway, the abutments and the adjacent ground should be taken extending out beyond any known flood heights. The size, location and grade on any gates should be obtained. A typical section showing the shape of the spillway should be made. The approximate distance of the dam from the bridge should be noted and all levels referred to the same datum as at the bridge site.

2. Other Bridges on the Stream

Any other bridge, up stream or down, that might affect the flow at the site should have its opening measured, and the grade of bed of stream and top of openings referred to the same datum as at the bridge site.

If the bridge is over any part of a lake or pond, cross sections should be taken across any dam or pond outlet, as detailed in Section 2.2.8(C)1 above.

3. Water Levels

Water levels on the date of survey and for a maximum flood should be obtained even though flood levels may be approximate. These levels should be taken immediately above and below the existing bridge and should be taken at each of the channel sections up and down stream, and beyond if necessary, to establish the top-of-water grade.

The flood water height at the crest of any controlling dam or at any adjacent bridge should also be taken.

If the bridge is over tide water, a tide gage should be set and continuous observations made over a period of a few days during normal tides. Observations should also be made as to whether the velocity of the water through the present opening seems excessive.

4. Character of the Stream

Comment by the survey party should also be made as to the character of the stream, the bed of the stream, and the banks

as follows:

Is the flow swift or moderate ?

Is there evidence of scour, either under the bridge or immediately below ?

Is there evidence of drift or debris in stream ?

Ice conditions (bad, moderate, or none) ?

Any evidence of harmful chemicals in stream?

Is bed of stream strewn with boulders, gravelly, sandy or muddy?

Are banks smooth, lined with trees, grass or boulders ?

Does surrounding land confine the stream or is it flat or swampy?

Does evidence indicate that the present size of the structure is inadequate?

Is the stream used for boating ?

2.3 CONSTRUCTION AND FINAL SURVEYS

2.3.1 Supervision and Administration

The Construction Division is responsible for the administration and supervision of construction projects undertaken by MHD. The responsibilities and duties of construction survey have evolved over the years and are detailed in MHD Standard Specifications, Section 5.07. The Resident Engineer is responsible for supervision of the construction work, and also for survey parties assigned to the project. While the Resident Engineer will assign the hours of employment and specific survey work, such survey parties also are responsible to the District Survey Engineer. The District Survey Engineer assigns and/or transfers parties between projects and exercises general supervision; he/she, or designee, will visit all projects periodically to ensure that proper surveying techniques are being followed and to give technical advice.

The Resident Engineer will be responsible for the time records of private parties and for endorsement of weekly reports. The Survey Engineer will process records of assignment, time sheets and payment vouchers.

Survey and Construction personnel must cooperate closely so that their respective operations will occur in the proper sequence in relation to the construction schedule. This schedule and all other project details including "basis of payment" items are the responsibility of the Resident Engineer. Close coordination is necessary to ensure that construction operations do not result in the destruction of survey markers.

2.3.2 Survey Work Performed on Construction Projects

Survey work on a construction project is performed by MHD or contract survey parties directed by the Resident Engineer, MHD Construction personnel, or the Contractor, each with specific responsibilities. Over the years, however, Survey's duties have tended to expand to include duties formerly performed by others.

The proper delineation of duties is:

A) Survey Section's Responsibilities:

* Establishment of baselines or centerlines of construction for main roadways, ramps, service roads, side streets and other major dry land items. Reproduction of baseline and centerlines, or lines offset to them when roadway cuts and fills have been completed. Levels may be taken on the points marking these lines.

* General bench mark control for the project.

* Original grade stakes at 20 m intervals.

* Preliminary and final surveys of pits (if borrow is paid by pit measure) and dredging areas, semifinal cross sections on ledge, peat, loam, etc.

* Control for structures, which shall consist of range lines on centerline of bearings or centerline of piers, face of abutments and wingwalls, horizontal and vertical control for beam seats along with bench marks close to structures for vertical control. Structures shall include but shall not be limited to bridges, culverts, dams, buildings and walls.

* Control for alignment of curbing or edging on ramps and at other complicated locations.

* Bound points and sideline stakes.

B) Resident Engineer's/Construction Section's Responsibilities:

* All necessary stakes for pipes and head walls; establish all catch basin and manhole locations as to line and grade.

* All necessary field checks on lines and grades established by the Contractor.

C) Contractor's Responsibilities:

* The Contractor shall employ qualified engineering personnel to insure adequate control and shall furnish and set stakes of the quality used by MHD for control staking. Rough stakes may be used to denote top and bottom of slopes, edge of pavement, gutter lines, etc.

* The Contractor shall furnish and set, at his own expense, all remaining stakes (such as batter boards, slope stakes, pins, offset stakes, etc.) required for the construction operations and he shall be solely responsible for the accuracy of the line and grade of all features of his work.

* The Contractor shall be held responsible for the preservation of all stakes and marks placed by or for Survey or Construction personnel. If any such stakes or marks are disturbed or destroyed by the Contractor, the cost of replacing them shall be deducted from the payment for the work as detailed in MHD Standard Specifications Section 9.05.

2.3.3 Preparation for Survey Work

The Survey Engineer should prepare for a construction project well in advance of the advertising date so that he/she will have sufficient time to collect needed data and to organize and begin survey activities.

As more construction plans are being prepared via computer aided design (CAD), the electronic files associated with the drawings may be in a suitable format to create a "stakeout" file for the electronic fieldbook (EFB). The Supervisor should obtain any available electronic data from the designer as part of his/her preparation.

He/she should obtain survey notebooks and other pertinent material such as layout plans, calculations and construction plans. He/she can assign parties to reestablish and mark up the baseline, locate and improve vertical control, place and mark offsets on grade stakes, take cross sections for construction, etc. Construction plans should be available so that he/she can locate the limits of construction, and make certain that points such as baseline ties, grade stakes, bench marks, etc., are beyond the construction area. Extra sets of plans, including cross sections, should be available after the bid opening.

Construction plans, especially those on major projects, often are quite complicated and should be thoroughly studied. Calculations should be commenced early and completely checked with discrepancies promptly resolved. The layout plans provide vital information on layout baselines, sidelines, MHB points, etc. Sideline data will have to be established, including exact location of bounds.

2.3.4 Survey Field Notes for Construction Work

When construction survey work is about to begin, the Survey Engineer will issue the preliminary survey notebooks pertaining to the job, as well as additional blank notebooks as needed, to the project survey engineer or crewchief.

Preliminary survey notes, tabulations and sketches used for design and the notes, sketches and calculations derived from construction operations are important survey records for the project. Also valuable are the horizontal and vertical control data and records of measurements that will be the basis for contract payments. These important records must be kept safe from damage or loss. When not in use, they should be locked in a fire resistant safe, if available.

If an EFB is being used to assist in the staking out, a manual notebook must be maintained as well for the reasons listed above.

Electronic data collector technology is not at a stage where the sketching can be recorded, stored safely, and retrieved reliably.

Notes in the books should be plain, accurate and thorough. A crewchief unfamiliar with the project should be able to pick up a notebook and proceed with the work. A construction engineer should be able to make his/her sketches and calculations for the project Black Book without difficulty. Those who are not specialists in engineering should be able to understand and interpret the notes.

On large projects, the Survey Engineer should keep an index of notebooks and all work therein on card files as are used in District Survey offices and all such records should be transferred to the District Office when the project ends. He/she should also keep a record of the various survey work performed.

Survey notes are legal evidence. Original information in the notebooks must not be erased. Revisions or corrections must be noted in such a way that the original is still legible, e.g. reference made to another page, or original lightly crossed out and new information added. All notes should be accompanied by identification of the survey party and date. Notes should be checked and initialed by the person doing the checking.

Baseline, layout and bound point notes and the like, may be added to preliminary survey notes in even numbered books, if there is room, or to new books. Similarly, level notes may be added to preliminary notes in odd numbered books, or in new books. The majority of the construction notes will be in level books. As two survey parties cannot work out of one book and the Resident Engineer cannot work with a book a survey party has in use and vice versa, there should be several books available on the project. Separate books are required for certain operations on large projects such as bridge surveys, borrow pits, certain types of excavations, and layout and bound points. Complete cross indexing of notebooks is important. Project notebooks will be used not only by Survey and Construction personnel who should be somewhat familiar with the contents of the various books, but also, for example, by Finals Section engineers, who will have no prior knowledge of the project.

2.3.5 Baselines

The Surveyor will be immediately concerned with the baselines. He/she should reproduce these, using the original ties (preliminary) in most cases, making all of the stationing plainly evident on the ground so that the baseline can be efficiently utilized.

The original long ties as well as short "hook" or "swing" ties that may already exist will generally be of temporary utility on the project since they are likely to obstruct construction operations.

These ties should be replaced by relatively indestructible long range ties ones that can be used for replacing the control at various stages of the project, as needed. Two sets of range ties crossing key points on the baseline are effective, as the baseline points can be quickly and accurately replaced with the aid of one or two transits and minimum taping. Reliance on a tape alone for ties can cause inaccurate results, especially if the tape is of a ribbon type and the ties are fairly long. While points can be replaced rapidly by use of an EDM, surveyors are reminded that ties are needed by others who may not have an EDM.

By referring to the construction plans, the surveyor can estimate a safe distance for ties, such as 5 or 10 m beyond slope limits for the closest point. If he/she does not yet have plans, as would often be the case before the advertising date, the limits of construction can be estimated and most long ties placed. The surveyor may be able to carefully tie in the baseline during the period from before the advertising date to the commencement of construction without being hampered by construction operations.

The surveyor should make certain there are good ties at PC, PI and PT locations as well as at control point on curve (POCs) and curve centers (CCs). There should be a sufficient number of points on tangent (POTs), which should usually be no farther apart than 150 m. Control points should be intervisible. Where possible, some surveyors prefer to run out ties at 90° to the baseline, or radially on curves. This method simplifies locating sidelines and bound points, setting offset lines, setting up curves, etc.

To protect and identify baseline points, an ample number of flagged witnesses, or "risers", with identification of the points plainly marked upon them, should be placed. Stationing should be carefully set and markings plainly written on stakes or printed on the pavement.

Where construction centerlines have to be run out on the ground, they should be calculated for closure to the baseline since they must fit both on paper and in the field. Additional calculations will likely be made, as the surveyor will design and figure ways to connect the various controls for efficient field layout.

2.3.6 Vertical Control

Bench mark (BM) control should be checked, and transferred or expanded as necessary at the same time baselines are being reestablished and tied in for construction. It is advisable to have the vertical control well established before construction begins and other activities tie up the survey parties.

The project surveyor will refer to the construction plans and check location and number of bench marks against construction details. He/she may find some have to be transferred as they will be endangered by construction operations and some may have to be added for convenience or insurance.

BMs should be witnessed and identified plainly and accurately. The stations, offsets and descriptions should be plainly indicated in the survey level notebooks.

Two BMs should be in place at critical locations such as at structures of importance, not only for construction convenience but so they can be checked one against the other. Use of temporary or poorly identified turning points as benches is discouraged. Strongly located turning points can provide additional level checks.

Bench marks which are located within the clearing and grubbing area on trees, roots or trunks, should be transferred before clearing operations. The loss of the weight of the tree may affect the elevation when the tree is cut. Other benches, such as those on boulders or ledge, could also be affected and should also be transferred.

Geodetic bench marks or any other survey marker should be preserved whenever possible. The MHD Boston Survey Office should be notified if any geodetic markers are at risk.

2.3.7 Grade Stakes

Grade stakes, also known as "construction stakes", or "side stakes", must be set prior to construction. They are stakes set in reference to the baseline station, usually placed in pairs, one left and one right, normal to the baseline and beyond the construction limits, at each 20 m station interval. An accurate horizontal measurement is taken with a steel tape between the station and the middle of the top of the stake, to the nearest 0.05 m, and levels are taken to the high corner of the stake to the nearest 0.01 m. These stakes provide horizontal and vertical control for construction operations. An example of highway grade staking is shown in Appendix A-6, Sample Survey Notes.

Although grade stakes do have great utility on a project, they

cannot be depended upon for all construction control. The surveyor will find it necessary to reestablish the baseline at different stages of construction, and will need to use bench marks for vertical control.

The type of grade stake used as well as the manner in which they are laid out will depend upon the project, e.g., a superhighway vs two lane road, or an urban vs rural location. The grade stake is usually a 2" X 4" X 24" wooden stake, but in places where a stake cannot be driven or where another device would have greater utility, a hub, crow's foot, or tag nail may be used.

Where possible, grade stakes should be set so that they are on the cross section (normal to the baseline), intervisible, where they are not apt to be destroyed and close enough to the construction so that they can be readily used. The stakes should be driven solidly into the ground about half their length, with the wide face toward the baseline.

Stakes should be placed 5 to 10 m beyond the slope line, depending upon the particular project. On a small reconstruction job, for example, the stake may have to be close to the construction because of local features such as curbing, lawns, walls, etc. On a large project, the stake may often be 10 m beyond slope limits and a little beyond clearing, where it should be safe from harm.

For setting out grade stakes normal to the baseline, a right angle prism may be used for good results on short offsets, and for longer distances on level terrain. After the first stake is set, left or right, the other stake may be set by sighting line through the first stake and the baseline station. If they are not intervisible, it may be necessary to use the prism in both directions and set line stakes behind each grade stake.

The preferred grade stake location is normal to the baseline, left and right. When an obstruction is encountered, the offset stakes may be skewed near 90◦, or two can be set on the same side of line.

Grade stakes that are set out during the initial stage of a large project may have to be placed so far out that they will have limited practical use for work close to the baseline. They will be used for determining locations in their vicinity, such as defining the sideline, edge of clearing area, slope limits, some drainage, etc.

Since they are not usually needed at once, elevations on grade stakes may be taken at a more convenient time such as after clearing operations have been completed. If more convenient, notes may be recorded in the staking book at the time elevations are taken.

After notes have been taken and checked, they should be given to the

Resident Engineer so the Stake Sheet can be completed.

2.3.8 Preliminary Cross Sections

The Survey Engineer or crewchief must ascertain at the initial stages of a construction project whether complete new preliminary cross sections will be needed or whether some new sections and some check sections will suffice.

Preliminary design sections, if taken originally by field methods, should generally be acceptable. Often there is a need to check, to extend or to take new levels.

If photogrammetric sections are to be used, they should be checked in the field to ensure that they are sufficiently accurate. Special attention should be given to areas in which it would be difficult to see the ground from the air because of obstructions such as high grass, brush, trees, water, etc., which might require that some new sections be taken.

Preliminary cross sections are used as a measurement basis for earth excavation, muck and peat, loam, unsuitable material and other construction payment items involving excavation or movement of materials.

Grid sections are useful for such items as ledge (Class A Rock Excavation), borrow and gravel, and work around structures.

Profiles are used for drainage, areas around structures, drives and so on, and for wider construction areas if depths are uniform or otherwise able to be calculated to needed accuracy.

Levels are also needed for profiles or cross sections for various excavation operations on construction projects. Since Survey personnel normally perform some of these operations while Construction personnel perform others, the Survey Engineer or crewchief must coordinate who will be doing which operation, and when it will be performed.

2.3.9 Cross Sections and Related Work

Several survey techniques such as normal cross sections, grid sections and profiles are utilized on construction projects as a basis for determining quantities, for checking preliminary design figures and for grade work.

As CAD technology improves, the Department may allow quantity measurements on construction to be taken using an electronic field book (EFB). For small areas, manual notes are most efficient. On

large projects or borrow pits, the surveyor should discuss with the Resident Engineer and the District Survey Engineer the best method to obtain the quantities. Survey measurements for quantities are usually for calculation of volumes although areas and linear dimensions are also calculated.

The Survey Engineer (and/or crewchief) and Resident Engineer must cooperate so survey work will be timely and in accordance with the construction schedule and the methods of measurements to be employed. Consideration must be given not only to the MHD Standard Specifications and Construction Manual but also to the project plans, specifications, special provisions, agreements among the parties, etc. Regardless of one's background, it is often difficult to anticipate all of a project's survey needs.

Cross section work will utilize the techniques described in Section 2.2.7, Cross Sections and Related Work, with modifications as needed.

2.3.10 Semifinal and Final Surveys

In order for the Resident Engineer and/or the MHD Finals Section to compute quantities of material measured by cross section, the "before" (prelims) and the "after" (finals) sections are needed. Both the preliminary and the final shots should extend to at least the distance that will be designated as "old ground" in the finals (abbreviated "O.G." in the notebook).

Normally, final cross sections are taken "over the footsteps" of the preliminary. If the original sections were taken at 20 m stations, the finals should be also. Preliminary askew or odd sections should also be repeated.

There are several Construction "pay quantities" and "methods of measurements" that eliminate some Survey work entirely. The Resident Engineer should inform the Survey Engineer or crewchief when work normally performed by Survey personnel will not be necessary because alternate means of obtaining necessary data or measurements are being utilized. An example of a deviation from the general procedure is for "cut and fill" sections: while Survey would normally expect to take cross sections for finals at cut sections or mixed cut and fill sections, it may or may not be necessary to cover fill sections.

On ledge sections and other sections, which may involve calculation for payment to some predetermined pay line, fully executed final cross sections, may not be required. Ledge, for example, involves a pay line at a specified distance below the finished pavement and shoulders plus a specified distance beyond a slope line on the

sides. Even though the prelims were taken on a grid, the finals in the roadway area can be covered by regular 20 m cross sections. However, grid sections are necessary on remaining ledge at slopes to old ground.

Many items on the project can be measured without cross sections. Profiles can often be utilized for driveways, areas around structures and culverts, and also with some materials such as dense graded crushed stone, gravel sub-base material, impervious soil borrow, sand loam, pavement material, etc. Construction personnel can measure and calculate such quantities with little or no Survey assistance. Area and linear measurements of quantities do not usually involve Survey, nor do measurements involving weight.

Surveyors assigned to construction operations should be familiar with the MHD Standard Specifications and Survey Manual so that conventional practices will be known. Discussion with the Resident Engineer will be needed so that Construction's needs for preliminary and final measurements are efficiently met.

2.3.11 Borrow Pits

Borrow is earth material such as topsoil, ordinary borrow, gravel, sand, etc., taken from outside of the project layout for use on the project.

The preferred method of measuring borrow is to measure it in place on the project. When that method is not practicable, the Resident Engineer may request Survey to measure the borrow pit. The general procedure in measuring a pit is to take elevations in a grid pattern after the pit is stripped of non-payment material and to take elevations over the same grid pattern after material has been removed.

Depending on conditions and types of materials, other methods usually not requiring Survey are available and described in the MHD Standard Specifications.

A) Preparation

Property lines should be well marked to prevent encroachment. This is the Contractor's responsibility. Pit limits should be defined and marked in a prominent manner.

The Resident Engineer and Survey Engineer should agree on procedures for clearing and stripping. It may be desirable to place some main control lines after clearing and before stripping and to have the pit stripped before the grid is laid out and sections taken. Stripping will be inside the pit limits, and

sections taken over them.

B) Notebooks and Sketches

A pit must have its own odd numbered notebook, to be used for all pit work, sketches, control work, cross sections, etc. A complete sketch and description of the pit should be entered into the notebook, showing its location in reference to highways, baselines, its ties, and other data. All horizontal and vertical work should be recorded.

C) Pit Baselines

The main baseline should be located longitudinally through the pit, parallel to side banks and normal to end banks. It should extend well beyond pit limits. It should be tied into a local road, if practicable.

If the main baseline has to be assumed, as it would be at an isolated location, magnetic bearings may be used for the assumed direction.

If a pit abuts or is close to the project, it should be tied into the project control (baseline) with a traverse that can be closed.

Baseline control for the pit should be well tied in so that it can be reproduced in the future.

D) Bench Marks

Two or more bench marks should be established at safe locations at the pit. If the pit abuts or is close to the project, the bench marks must be on the same datum as the project vertical control. At least two bench marks on the project should be used. If the project is not close enough to the pit other bench marks may be used.

Where there are no known bench marks available, an elevation may be assumed. The assumed elevation should be a high enough number so that no pit elevations will be less than zero.

Leveling should be run highway style, closed and adjusted.

2.3.12 Bound Points

The Survey Engineer should begin computations during the initial stage of the project and set bound points as time and construction conditions allow.

The baseline, control traverses and bound point data should be on an accurate coordinate system. On small projects the crewchief can usually make needed computations with a hand calculator, but most coordinates for larger projects should be available.

A complete record of the bound work including the location of all bounds, dates set and by whom, angles and distances used to set them, all ties, etc., should be entered in an even numbered notebook.

As bounds are permanent witnesses to the legal layout as well as an aid in reproducing the baseline in the future, they should be accurately and carefully located on the ground. Bounds are defined in position on the layout plan by station, bearing and distance, or by coordinate value. Although they are mathematically accurate, they may not be ideally located from Survey's point of view. Although it may involve many calculations, the surveyor may desire to set the bound points "indirectly", from a nearby control point on the baseline or from a control traverse point close to the bound.

Bound points, especially those set indirectly, should be checked independently of the method used to set them. Checks can be made to other points, to the baseline, or to another bound.

A bound point set by Survey is usually a hub with a small nail or e-pin, with the hub set solidly into the ground. Close ties 2 m or so from the point may consist of hook ties to stakes or trees. Use of straddle stakes, from which string lines may be run to cross over the point, are usually preferred by the worker actually setting the bound. Strong, long ties should also be utilized to enable crew members to check or relocate the point.

Tie distances should be written on stakes and other ties. A witness stake, with bound identification written on it, should also be set.

"Pinning" a bound usually is done when the project is nearly completed and after the bound has had a chance to settle. An escutcheon pin is set in lead in the drilled hole on the bound and is an indication that the surveyors have made a final check on the point and that the location is accurate.

The bound should never be pinned from short ties. The surveyors should run the bound point in again from the baseline or from good control and check the work against the long ties.

2.3.13 Culvert and Bridge Surveys

Section 2.3.2, Survey Work Performed on Construction Projects, details work to be performed by Survey and Construction personnel and by the Contractor. It is sometimes necessary for Survey personnel, at the direction of the Resident Engineer, to perform work normally considered Construction's duties. Survey should not, however, perform work that is the Contractor's responsibility.

Survey should furnish necessary staking for pipes and head walls and establish line and grade for catch basins and manholes. Survey may also be directed by the Resident Engineer to check control work done by the Contractor.

Section 5.07 (E) of the MHD Standard Specifications states:

> Control for structures, which shall consist of range lines on centerline of bearings or centerline of piers, face of abutments and wingwalls, horizontal and vertical control for beam seats, along with bench marks close to structures for vertical control.

In many cases the surveyor will have to extend lines for a considerable distance to avoid destruction of control from construction activity.

2.3.14 Bridge Staking

Starting with the baseline, centerline of construction, or whatever line is used on the bridge plans to locate the structure:

a) Stake out, range and tie in the centerline of each abutment and pier.

b) Stake out, range and tie the concrete line of the wingwalls. Place a stake at the intersection of the concrete line of wingwall and face of abutment.

c) Stake the position of each outside beam, measure space and compare with plans.

d) If structure is not at right angles, compute square span from centerline of bearing to centerline of bearing. Check-measure these as staked out.

e) Measure distance from concrete line of wingwall to concrete line of wingwall along line of face of abutment and compare with plans.

f) Measure overall length of piers and compare with plans. Make a sketch showing how the structure was laid out, ranged and tied. Start with baseline measurements that were used to arrive at plan stations. Enter the information in an even numbered notebook, with check measurements shown on the sketch.

2.4 SURVEYS FOR PHOTOGRAMMETRY

The Department generally does two types of surveys for photogrammetry projects, a survey to establish horizontal and vertical control and a field-check survey to determine the accuracy of a mapping submittal. The control should be established first to at least Third-Order standards, then the photo check points should be obtained from the adjusted control.

Cloth targets or targets painted on the ground are the desired means of controlling aerial mapping. They provide a sharp, well-defined point and do not require photo interpretation by the surveyor in the field. Disadvantages to targets placed prior to flying are they may be removed or destroyed prior to flying, may fall in shadows when flown, take additional time and more points than necessary must be placed to ensure enough for each model.

Common practice is to have the project flown when conditions are optimum for the photography and then have the Photogrammetrist select suitable points from the photography. In this case the surveyor is given a set of contact prints with selected points, pin-pricked and circled on the back, with a description. Some points may require either a horizontal or a vertical position and some will require both.

More discrepancies in ground control arise from incorrect identification of points on the photograph than from erroneous survey data. If there is any doubt in the surveyor's mind as to whether the point selected on the photograph can be identified on the ground, has been disturbed, or is otherwise undesirable for use as a control point, another nearby point which can be clearly identified on the ground and on the photograph should be selected. When an alternate point is selected, the surveyor must note the new description on the photograph and in the fieldbook, and provide enough details to eliminate any confusion.

2.4.1 Field Procedures for Photogrammetry Control

The ground control survey must be accurate if the photogrammetry is to be accurate. While higher order surveys are preferable, Third-Order traverses and levels are usually adequate. Whenever possible, horizontal control points to be used for the mapping should be incorporated into the traverse. Other required horizontal positions may be obtained by side shots provided there is suitable redundancy to ensure accuracy. A minimum observation would be two complete sets of two readings, one direct and one reverse on the backsight and the same on the side shot in each set, with a mean of five EDM distances in the direct position and five in the reverse.

Vertical control point elevations should be obtained using level and rod only. Prior approval of the MHD Boston Survey Office and the Photogrammetry Engineer is required for use of a total station to establish control elevations electronically.

2.4.2 Field Procedures for Checking Photogrammetry

The mapping that is generated based on the control points established in the above procedure will need to be checked for accuracy. The purpose of this field verification is to determine if the stereomodels have been set up properly and the mapping compiled accurately. Objects or selected points that are plotted on the mapping must be verified horizontally and vertically.

The surveyor will be provided with a set of preliminary prints of the mapping. The number and spacing of check points will be designated although the surveyor will have some leeway in choosing locations to expedite the work. Checking vertical positions by level and rod and checking horizontal positions by tape usually accomplish this work. Because most MHD photogrammetric mapping is oriented along highway corridors, it is flown and plotted in strips where lateral dimension checking can be done by taping relatively fast.

The locations checked and the results should be noted in red on the prints and in the fieldbook and returned to the MHD Boston Survey Office so that a table of field results can be compiled.

SECTION THREE

SURVEY TECHNIQUES

3.1 CONVENTIONAL FIELD SURVEY EQUIPMENT

3.1.1 Capsule Description of Conventional (non-Global Positioning System) Surveying Instruments

a) Theodolites/Transits

These instruments are primarily to measure horizontal and vertical (zenith) angles and to establish line and grade. While the major difference between the instruments is that the reading system is external on a transit and internal on a theodolite, both instruments perform similar functions and will be collectively referred to as theodolites. Horizontal angles are derived from horizontal circle readings and vertical angles from vertical circle readings. Stadia hairs, used in conjunction with a leveling rod, may be used to obtain horizontal distances and elevation changes. Electronic theodolites have digital readout, which relieves the surveyor from reading scales or micrometers.

b) Electronic Distance Measurement Instruments (EDMs)

An EDM derives a slope distance from the instrument to a reflector by measuring the outgoing and incoming wavelength of various electronic sources. An EDM is generally used in conjunction with a theodolite to derive horizontal distance and elevation change from the slope distance and zenith angle. Many EDMs mount directly onto theodolites. While some newer EDMs can measure a distance to an object over short distances (less than 50 m.) without the use of a prism, use of a prism is required for all MHD work.

c) Levels

The level is used to determine differences in elevation; its line of sight is fixed at horizontal when the instrument is leveled. When a leveling rod is observed, the elevation change from the instrument's line of sight down to the bottom of the rod can be obtained. Levels are often equipped with stadia hairs for distance measurement and often have a very rough external scale for horizontal angle measurement.

There are three basic types of levels: spirit, compensator and electronic. Electronic levels have a compensator and utilize digital readout to read a barcoded invar level rod to determine elevation differences and distances from instrument to rod.

d) Tapes

A tape can be of cloth, fiberglass or steel construction and is used to measure horizontal distance between two points.

e) Total Stations

This instrument is an integrated theodolite/EDM with digital readout of horizontal circle, zenith circle, and slope distance. Most total stations have a variety of on-board software for computation of horizontal angle, horizontal distance, elevation change, coordinates, and coordinate inversing. The total station has serial communications so an electronic field book (EFB) can automatically store desired information and perform some instrument commands such as zeroing out the horizontal circle. Some instruments have EFB on-board capabilities with data storage primarily on PC-MIA compatible cards, very durable hard disks which are the equivalent of structurally-sound floppy disks and are a rugged mechanism for data storage under field conditions.

3.1.2 Detailed Description and Applications for Conventional (Non-Global Positioning System) Surveying Instruments

A) Theodolites

This discussion on theodolites is also applicable to transits, which are similar instruments, and to total stations, which contain a theodolite as an integral part.

Most theodolites require a tripod different than that used for transits. The type of tripod used for levels is also suitable for most theodolites.

A tripod is a three-legged device; the legs are usually constructed of wood or aluminum. The end of the legs contains a pointed device connected to a piece of metal which one can step on to secure the legs by pushing them into the earth. In cases where the legs cannot be pushed into the earth (such as on a concrete surface), care must be taken to continuously monitor the stability of the instrument. Sandbags or similar items can be used to secure the bottoms of the tripod legs or devices such as tripod triangles or chains can be used to ensure stability. Wooden footpads can be used to prevent settlement on bituminous or similar surfaces.

The tripod has a flat mounting surface at its top to which the bottom plate of the theodolite is secured via a knurled threaded knob on the tripod. The knob threads directly into the bottom of

the theodolite's tribrach base for a secure fit between the two devices. Tribrachs, which are detachable from the theodolite, are very useful in "leap frogging" in traverse operations, a procedure that eliminates many problems with measurement of height of instrument and target.

Tripods either have fixed length legs or adjustable legs, which can be withdrawn or extended. While the former ensures stability, the latter is more commonly used because it is adaptable to different instrument heights, undulating ground surface and efficiency in setting up over a point. Twist knobs are one of the standard mechanisms for release and tightening the devices, which allow legs to be extended or contracted. Tripods are approximately 1 meter long when contracted and 2 meters long when fully extended.

A theodolite often has to be set up so its vertical axis is directly over a survey point when the instrument is level. On older instruments this was accomplished using a plumb bob, which was attached to a hook set directly below the theodolite. The optical plummet has generally replaced this method, which is now standard for most instruments. The optical plummet is a mechanism, which allows the surveyor a visual line-of-sight vertically down from the center of a leveled instrument. A right angle prism is usually part of the optical plummet system so that the surveyor can look horizontally through an eyepiece and actually see vertically down from the center of the instrument to the survey point. A precise pointing mechanism, usually a bull's eye circular cross hair, is part of the viewing system of the optical plummet. The cross hair can be focused separately from the ground. The same leveling and setup principals apply to prisms and targets mounted on tribrachs and tripods. For targeting systems, which do not have a vial bubble, one must rely solely on a bull's eye bubble; its proper adjustment is critical.

A theodolite's two functions are to measure horizontal and zenith (vertical) angles. The horizontal angle is derived from the difference between two horizontal circle readings of two independent lines and the zenith angle is derived from zero on the vertical circle being at the zenith of a leveled instrument. Some theodolites measure vertical (above or below a horizontal line) angles instead of zenith (down from straight up) angles. While the angular readout can be in grads and/or mils, most instruments have angle measuring systems, which are degrees-minutes-seconds, based. Decimal degrees are not standard in surveying practice except as an intermediate value for computational purposes.

A viewing system for pointing the instrument is usually a telescope with a viewing focus, which rotates both vertically and

horizontally about the intersection of the instrument's horizontal and vertical axes. Horizontal and vertical cross hairs, which should have a separate focus device, are used for precise pointing. The vertical cross hair usually contains stadia hairs for estimating horizontal distance and elevation change when used with a level rod and zenith angle.

There are several methods to determine the angular accuracy obtainable from a theodolite or total station, the least reliable of which is the Least Count method. The least count of a theodolite or total station is that value that can be read or recorded without interpolation. This was a reasonable determination for older vernier-type transits but it is inadequate for modern theodolites. Certain scale reading theodolites have a least count of 1' but can be easily and accurately interpolated to 0.1' and often have a standard deviation on one set of angles of under 3". Other theodolites may have a least count of 1" but are found to have a Standard Deviation (SD) of about 5".

An indicator of an instrument's accuracy that is usually reliable is the accuracy rating provided by the manufacturer. The owner's manual for most instruments lists an angular Standard Deviation, which the instrument is capable of achieving. This value is based on the International DIN specifications, which have standardized tests and procedures for rating theodolites and total stations.

Probably the best method is the field test that determines the instrument's Smallest Measuring Ability (SMA). The instrument's SMA is obtained by taking repeated measurements over lines of 200-400 meters in length. The test should include four well defined sights, with at least four direct and four reverse readings and repeated at least three times. The standard errors for the three tests should be relatively consistent, or testing must continue until consistency is achieved. The standard deviation becomes the instrument's SMA. The least count for an instrument could be smaller or greater than its SMA, depending on how the least count relates to the internal optics and motions of the instrument. SMA is similar to the DIN specifications for theodolites and total stations but with less rigidity in the testing process.

The theodolite contains tangent (motion) screws for both horizontal and vertical pointing. When loosened horizontally, the instrument revolves around its vertical axis freely. The telescope freely rotates vertically when the vertical motion is loosened. When secure, each motion has precise pointing mechanisms called slow motion tangent screws. These are usually "built in" components of the general tangent screws.

Repeating theodolites are the most common type. American-style transits are repeating instruments as are some total stations. Repeating, or doubling angles is an important procedure when an instrument has a high SMA. To get a precise reading, angles turned should be added and calculated so that the final value is obtained by dividing the sum by the number of times the angle was observed. This is the best method when an instrument's SMA is 10" or above.

Directional theodolites differ from repeating instruments in that there is no lower motion so that angles are being measured continually. Most precise theodolites and total stations are directional instruments although some total stations have modes, which enable angles to be summed. Most instruments and data collectors are set up for directional angular input. One angle or set is defined as the average of one direct and one reverse angular observation. Project needs dictate how many sets of angles must be observed.

With modern total stations and electronic fieldbooks, the need for a lower motion is not critical and has been eliminated on many instruments. Likewise, many precise older theodolites were called positional or directional instruments where horizontal angles were derived from the circle readings and were not supposed to be used for layout or angle doubling. These instruments were designed for control traversing where zeroing out an instrument or retaining an angular value were not critical concerns.

Leveling a theodolite is accomplished by using three leveling screws, which are usually between the bottom plate of the tribrach, which attaches to the tripod and the rest of the theodolite. Most instruments contain a bull's eye bubble for rough leveling and 1 or 2 alidade bubbles for more precise leveling. General practice is to level the bull's eye bubble and then the alidade bubble(s). Turning the leveling tangent screws means the vertical axis of the instrument will be moving off the point on which it is centered.

Many newer theodolites and total stations have axis compensators, which will attempt to keep an instrument leveled once manual procedures have been performed.

Nearly all theodolites allow the instrument's scope to be rotated in a complete vertical circle. Considering that an instrument's vertical circle is zenith-based with zero on the reading scale straight up, in one direction downwards (direct or circle left) the scale reads from zero to 180◦ (straight down) while in the other direction (reverse, indirect, or circle right) the scale

reads from 360◦ (equivalent to zero) down to 180◦ degrees straight down.

Measuring horizontal angles by averaging an equal number of times in direct and reverse eliminates both vertical and horizontal collimation errors. The same is true for zenith angles except a simple average cannot be used because the correction will be of the same sign to both direct and reverse readings. In a perfectly collimated instrument, the sum of a direct and reverse vertical reading will be 360◦00'00". As an example, precise direct and reverse pointings result in zenith angle readings of 89◦59'20" and 270◦00'20" respectively. The sum is 359◦59'40" and indicates ten seconds should be added to each reading.

B) Electronic Distance Measuring Instruments (EDMs)

EDMs, developed from experimentation by physicists trying to measure the speed of light, have revolutionized surveying since for most purposes the tedious job of measuring distance in tape lengths is no longer required. In addition, combining an EDM with a theodolite's zenith angle produces both horizontal distance and elevation change from the instrument's center to the reflector. Measuring height of instrument and height of reflector above occupied points enables the aforementioned elevation change to be converted to an elevation change from instrument-occupied survey ground point to reflector-occupied ground point, often referred to as a "mark to mark" elevation change.

A sequence of mathematical operations may be required for reduction of a slope distance/zenith angle in combination with height of instrument/height of reflector. The slope distance may require correction for temperature, pressure and combined instrument/prism offset. The four values are used in conjunction with earth curvature and atmospheric refraction correction to obtain a horizontal distance and a mark-to-mark elevation difference. The horizontal distance may require reduction to the ellipsoid and a scale factor applied to obtain a grid (state plane) distance.

Care must be taken if the EDMs vertical axis is offset from the theodolite's similar axis. This requires an instrument-offset correction, which needs to be applied to all measured slope distances.

It is also necessary to consider that the distance is measured from the EDM while the zenith angle is measured from the theodolite, which is usually below it. Since the distance from the EDM to the theodolite is very small, it usually only requires attention on short lines (less than 30 meters) which have unusual

zenith angles (deviations from horizontal of more than 30 degrees). On most total stations the EDM and theodolite center are vertically coincident and thus this offset is eliminated. The height of instrument should always be measured to the optical center of the theodolite, as that is where the zenith angle is measured.

EDM-produced slope distances are affected by temperature and pressure. "Corrections" for these variables are scale factors, which are multipliers to the measured value and are so near a value of one that lines less than 200 m in length are usually affected less than 0.001 m. The correction can be considered insignificant when measured lines are shorter than 200 m. Before an EDM is used, one should test it with some temperature and pressure extremes to see when the corrections become significant. In control traversing, it is imperative that correct temperature and pressure values are entered and that atmospheric corrections are made.

The corrections are computed through equation or vendor-supplied nomographs which require user interpolation. Most total stations allow field-entered correction values so that the distance measured will already be corrected. Some total stations now have on-board thermometers and pressure gauges, which communicate directly with the EDM so the user does not have to enter any correction values.

EDMs require a prism at the point to which the measurement is taken. Standard prisms are approximately 0.07 m in diameter while "peanut" prisms, used for some short line measurements, are approximately 0.02 m in diameter. In control traversing, long lines may require the use of more than one prism.

Larger prisms are secured into a holder, which mounts on a prism pole, or into a tribrach, which can be mounted on a tripod. The holder usually can secure the prism with a zero offset or a 0.03 m offset. It is of utmost importance that the combined affect of EDM instrument offset plus the prism offset be properly determined. Most newer model EDMs have internal dip switches or dials which enable input of the combined offset correction.

Once the combined offset has been entered, it is important to check its integrity. A distance of approximately 30 m should be taped on flat ground as a baseline comparison to the EDM-measured value. If these two values are in disagreement beyond random error limits (more than 0.005 m) it is likely that the wrong correction was dialed into the EDM and needs to be corrected. Offsets can change by a small amount just because of everyday jostling of equipment.

Prism poles experience a significant amount of hard field use and should be checked routinely by this baseline process. A significant departure from the taped value would indicate the bull's eye bubble on the prism pole is in need of adjustment.

It is useful to have a well-monumented baseline readily available for checking EDMs. It is also important to take an EDM to the nearest calibration baseline on a routine basis (2-4 times per year) to check its measuring ability over a variety of line lengths. Tribrachs and optical plummets should be calibrated before the baseline is visited. (See Appendix A-8, Massachusetts Calibration Baselines).

If prisms and tribrachs are interchanged between crews and/or total stations, it is extremely important to perform a quick distance comparison to a taped distance. The most common error in EDM use is incorrect prism offset.

C) Levels

The level is the standard instrument for determining elevation differences, especially where the accuracy derived from trigonometric means may not be sufficient.

Most levels are defined as either spirit or compensator (also called "automatic"). The line of sight for a spirit level is determined by leveling a tubular spirit level attached to the instrument. The sensitivity of the liquid bubble determines to a large degree the precision of the instrument. There are many different models depending on the precision and accuracy desired.

The controlling mechanism on Dumpy and Wye levels, as on a transit telescope, is a spirit level. The instrument used for most of the first-order level runs performed by United States Coast and Geodetic Survey during the early to mid 1900s was a Fischer Level, a Dumpy Level modified for that purpose.

The Spirit Level has generally been replaced by the Automatic (or Compensator) Level because of its simpler operation and easier-obtained accuracy.

On an Automatic Level the line of sight is automatically kept horizontal by the compensator, a set of prisms or mirrors that swing freely in response to gravity. The compensator may swing from wires or be attached to a pendulum or flexible spring. The cost and accuracy of Automatic Levels cover a wide range from basic construction levels to sophisticated first-order levels with reversible compensators and built-in parallel plate micrometers.

Most precise leveling performed since the 1950s was performed with automatic levels equipped with a built-in or add-on parallel plate micrometer. The micrometer attaches to the objective lens of the instrument and permits the level rod to be read more precisely. This increases the accuracy attainable by any level and enables most levels to be used for precise leveling applications.

A theodolite or total station can be used as a level provided the collimation of the instrument is correct, that is the line of sight is level when the zenith angle is 90◦00'00" and 270◦00'00".

Levels are generally not set precisely over survey points and thus have no optical plummet. Most levels have the capability for use of a plumb bob for rough point centering. Levels mount on tripods in the same manner as theodolites. The telescope on a level can only rotate horizontally because preservation of a horizontal line of sight is critical. The telescope has both an image and a cross hair focus. Most levels have a horizontal slow-motion tangent screw similar to that on theodolites. The vertical cross hair contains a distinct center position with upper and lower stadia hairs. The stadia constant is usually 100 but should always be checked. The stadia hairs are utilized to check the middle cross hair reading and to estimate the distance from instrument to the level rod.

Rods used for conventional leveling are usually of wood construction with a metal strip attached to the face with either centimeter or half-centimeter graduations. The rods can be comprised of either two or three sections with a total length of either 3, 4 or 5 meters. Telescoping fiberglass rods as long as 7 meters are available but these are primarily used for the lowest precision work.

Standard level rods have direct reading ability generally between 0.001 m - 0.01 m depending on the type of rod being used for a particular application. Interpolation between least count can be estimated by observation or can be improved with the use of a vernier system, which attaches to the rod. Level rods are approximately 2 m in length contracted and 4 m in length when extended. A level rod should be vertical when being read, a procedure made easier by attaching a bull's eye bubble called a rod level. When a rod level bubble is not available the rod can be rocked back and forth in the direction of the level with the instrument operator reading the smallest value as the vertical reading.

The level rod is an important part of the leveling process. The most precise leveling instrument cannot provide high-order control unless matched with a proper rod. Most precise

applications require a one piece invar leveling staff, usually 3 m in length, of which there are two types, single scale and double scale. The scale on single scale rods consists of a delicate invar strip with 0.005 or 0.01 m graduations. Double scale rods are the most precise. They have scales on both sides of the rod, one side set at 0.0 (low scale) and the other side set at a known offset (high scale). While high accuracies are attainable on a single run using specific leveling procedures, FGCC First-Order, Class 2 and Second-Order specifications usually require double runs. Single-run lines may be acceptable for some First-Order, Class 2 and Second-Order applications if the invar rods have different offset values between high and low scales. Having different scale differences allows detection of the most common error in precise leveling, switching backsight and foresight readings. Other important components of a precise leveling process include portable turning points (plates or drivable pins) and braces to keep the rod vertical.

For those levels, which read rods automatically, the invar staff is precisely bar coded. It can only be read under conditions conducive to a normal user's visual reading of a level rod. This type of level contains on-board software for data collection and also has the capability to connect with an external electronic fieldbook system.

D) Tapes

Prior to the development of EDMs, the tape was the standard means for precise distance measurement. While many standard field procedures now use the EDM for distance measurement due to its efficiency, certain field operations still warrant the use of a tape. An example is measurement of bridge components less than 30 m in length. Steel tapes are used for precise measurements of short lines, where they provide a more accurate determination than an EDM or total station.

Steel and coated steel tapes are standards for precise measurement because of their durable nature. Cloth and fiberglass tapes are much easier to maintain and are used primarily where high accuracy is not a concern. A typical steel tape is 30 m in length with a least count of 0.001-0.005 m. Cloth tapes vary in length but rarely exceed 30 m due to comparatively high elasticity.

Any tape needs to be routinely checked against some measured baseline because normal use will eventually affect its length. Tapes are also subject to the systematic errors of temperature, pull and sag. These errors require correction, if deemed significant in size, for a given measured length. On short lines (less than 15 m) the systematic errors are generally smaller than

the survey accuracy requirements.

E) Total Stations

A total station is designed as an integrated theodolite/EDM system. This means the EDM is not mounted on the theodolite and, on most total stations, the EDM is co-incident with the theodolite's line of sight.

The above discussion on theodolites and EDMs applies directly to total stations in terms of components and measuring system clarification. The slope distance, zenith circle, and horizontal circle are all provided by digital readout.

All total stations come with on-board software, which is accessed through buttons and keyboards. Typical on-board software provides for zeroing of the horizontal circle, reducing slope distance and zenith angle to its horizontal and vertical components, calculation of a remote distance, identifying angle and distance repetition errors, and calculating coordinates as measurements are made. Care must be taken since these calculated quantities are often derived from single measurements, which may not be corrected for systematic errors, and also that coordinate calculations have a consistent directional orientation defined.

All modern total stations have the capability to interact with a data collector or electronic fieldbook (EFB). Manufacturers often provide their own EFB systems or third party EFB systems can be utilized with a variety of total stations. Some total stations have integral on-board EFB systems, which eliminate the need for a serial communication cable and external EFB hardware and software. PC-MIA cards are becoming the standard for EFB storage of the collected field information or coordinates provided from the office computer for layout purposes.

3.2 FIELD OPERATIONS WITH CONVENTIONAL SURVEY EQUIPMENT

MHD control surveys follow Federal Geodetic Control Committee (FGCC) guidelines (See Appendix A-11) for procedures, instrumentation and monumentation.

Different job requirements dictate different field equipment and survey procedures. The description of a point and its 3-D coordinates are the desired data for some types of survey. In route surveying, it is necessary to relate a point to a defined centerline and to have much of the collected data linked to a defined project centerline by station and offset.

3.2.1 - Capsule Description of Some Types of Surveys

a) Traverse

Any form of traversing is a control-type survey without the control survey's high-accuracy requirements. Traversing is loosely defined as a procedure for obtaining 2-D or 3-D coordinates for survey stations where vertical data is not necessarily desired. Traversing, along with GPS, often provides the coordinate base necessary for surveys, which produce topographic information.

b) Differential Leveling

The use of a level for control-type surveys determines the elevations of points, which will later be used for topographic purposes. While traversing and GPS can produce elevations, they are often not of sufficient accuracy. Differential leveling is the proven means of obtaining highly accurate elevations.

c) Topographic Data Collection

The elevations of topographic points and lines are determined and collected and may or may not be used in a ground model. Attributing of this information is especially important in linking the data to the proper symbol from the electronic plan's library of symbols. Additional attributing can describe the significance of the point (tree type, diameter, crown width, etc.) and must appear as text information in topographic representation.

d) Digital Terrain Model Data Collection

Spot elevation and break line data is collected for the generation of a Triangulated Irregular Network (TIN), which produces the Digital Terrain Model (DTM) of the ground surface.

The DTM is extremely useful as any type of elevation information (contours, profiles, cross sections, 3-D views, etc.) can be generated from it.

e) Cross Section Data Collection

If a DTM or contours are not desired products but a route design will require some volume calculations, collection of cross section information at defined intervals along a centerline(s) can be desired survey information. Various procedures can be used to collect this information.

f) Stakeout

As opposed to survey data collection, stakeout involves the field layout of specified points, which were calculated in the office. These could include lost or new boundary locations, centerline or right-of-way points, building layout, etc. If elevation is a desired product, one must be able to locate the desired elevation (up or down) relative to the surveyed point (e.g., cut or fill elevations on grade stakes).

3.2.2 Detailed Description and Applications for Some Types of Surveys

A) Traverse

Traversing is the usual form of conventional survey control extension. Traverses are simply interconnected lines of distance and angle measurements.

Most MHD traverses start and end on pairs of geodetic control points of known state plane coordinate values. The initial and terminal bearing or azimuth are computed from a grid inverse on the control points. Some traverses may start or end on a single point which has a known grid azimuth to a reference object (spire, water tank, tower, etc.). Any other control points, which lay in or near the path of the traverse, should be incorporated into it.

The two primary forms of traverse are the loop (closed) and the link (open ended). The loop begins and ends at the same point of known coordinates. The initial bearing for the traverse can be derived from coordinate inverse, through astronomic observation, or in rare cases is assumed (a practice not recommended). The link traverse starts and ends at two different points of known coordinates. Determination of the starting bearing can be similar to that of the loop, or it can be derived from the bearing between the known coordinates of the endpoints. Computationally,

this last procedure requires starting on an assumed bearing. When the end station is reached, the traverse can be rotated to the bearing defined by the link traverse endpoints although MHD does not recommended the practice. Since simple link and loop traverses can close well despite systematic errors, they should be checked carefully.

In route surveys, the control traverse often becomes a series of interconnected link and loop traverses. The surveyor should make additional angle and distance measurements between intervisible stations whenever possible to increase the number of checks on the accuracy of the survey and add more geometry to the survey "network." The traverse should tie into all control points in the area so that the integrity of the existing coordinates is verified.

When traversing, prisms should always be mounted on tribrachs rather than prism poles to ensure quality measurements. Whenever possible, a prism should be mounted on a tribrach on all backsights. This provides a check on the change in distance and elevation shot when the backsight station was occupied and also when the presently occupied station was foresighted. It also eliminates some environmental systematic errors when the elevation difference measured in opposite directions is averaged.

If elevation is a desired product of the traverse, it is critical to measure precise heights of instruments and heights of targets at all setups and sighted stations. It is also important to sight prisms precisely, both vertically and horizontally, and to ensure that the instrument's vertical collimation is correct and level at 90◦00'00" and 270◦00'00".

Theoretically sound practice is to measure both angles and distances an equal number of times in the direct and reverse (D&R) positions and then to average. This eliminates the instrument's horizontal collimation errors. The average of a D&R reading is considered one repetition or set.

Requirements for a particular job will dictate needed accuracies for the angles and distances, an acceptable least count for an instrument, the number of repetitions required, closure limits/precisions, monumentation, etc. A common angular accuracy requirement is a standard deviation (SD) of 3" for one repetition; a common distance accuracy requirement is 5 mm +/-5 ppm for one measurement.

Typical specifications for an MHD project requiring Third- Order, Class I horizontal and vertical control might include the following:

1. Control:

   a) Horizontal - A minimum of four MHD or NGS Second-Order, Class II (or above) control points, with two pairs intervisible.

   b) Vertical - A minimum of three MHD or NGS Second-Order, Class II (or above) control points.

2. Monumentation:

   a) Horizontal: All traverse points must be stable, flush, semi-permanent marks with at least 40% being 1-meter iron rods 0.015 m in diameter, driven flush to the ground.

   b) Vertical: A bench mark every 500 m consisting of a semi-permanent mark not subject to movement.

3. Instrumentation:

   a) Horizontal and vertical angle instrument having a least count of 1" and able to produce a SD of 3" in one set of D&R readings.

   b) Distance: EDM with a SD of at least 0.5 mm +/-5 ppm per measurement.

   c) Elevation: Level capable of leveling a 1 km line to a SD of 1.5 mm for a double run.

4. Field Procedures:

   a) Horizontal Angles: four sets D&R, each set within +/-5” of mean.

   b) Zenith Angles: 2 reciprocal sets to agree within 10".

   c) Distance: Reciprocal, at least four in each direction to agree within 10 mm. If distance is obtained directly from instrument, Zenith Angle must be correct to within 10".

   d) Levels: 3-wire single run between control, single hair to level traverse points.

5. Closures:

Azimuth - 10"√(No. of lines in traverse)
Position - 1/10,000 after azimuth adjustment
Vertical - 0.012 m√(Dist in km of line between control BM)

6. Final Products:

Copies of all control used with published values, description and sketches of all points established, copies of traverse and level adjustments and a listing of state plane coordinates on NAD 83 and elevations based on NAVD 88 for all traverse points and bench marks established.

B) Differential Leveling

A level and level rod are commonly used to determine elevation differences between survey points. By tying elevation differences to one or more points of known elevation (bench marks), the elevation of points can be determined by differential leveling.

The quality of the level in terms of maintaining a horizontal line of sight, the magnification power of the telescope, and keeping the rod vertical all play a role in the accuracy of elevation determinations. The requirements for a job should include parameters such as these in addition to desired closures as a function of the number of setups or distance leveled in kilometers.

Differential leveling is particularly sensitive to systematic errors of non-horizontal line of sight and environmental conditions of earth curvature and atmospheric refraction. Balancing backsight (BS) and foresight (FS) distances during the leveling process eliminates most of these errors. All control leveling above FGCC Third-Order requires an invar one-piece rod. Virtually all precise leveling should be done with a compensator level equipped with a parallel plate micrometer and invar leveling staffs.

For control purposes, standard differential leveling field procedures normally require that all three wires (upper, middle, and lower cross hairs) be read and meaned. This allows a quick check that the rod intercept difference of the upper and middle readings compared to the interval of the middle and lower cross hair readings indicates reasonable closure. It also allows computation of the distance from the level to the rod by multiplying the difference between the upper and lower cross hair readings by the stadia constant. Once a backsight is chosen, the foresight should be paced so that its distance will approximate the backsight distance as closely as possible. The backsight and

foresight readings should be taken as close together as possible time-wise to eliminate changes caused by atmospheric conditions and settlement. Productivity and precision are enhanced by use of two level rods. Keeping the level vertical when moving between setups also enhances data quality. The backsight and foresight distances should be summed throughout the leveling process. Any setup which has an imbalance between BS and FS greater than 10 m shall be redone immediately to bring the distances closer together.

As a level line nears completion, the sum of the backsight distances should be compared to the sum of the foresight distances. Instead of matching a backsight distance to a foresight distance, the surveyor should balance the sum of the backsights as nearly as practicable with the sum of the foresights.

Correction for non-horizontal line of sight requires the use of a peg test (described in Section 3.4.2 (D), Differential Leveling) to determine the amount of departure (elevation or depression) from horizontal per 100 m of sight distance. A peg test should be performed before and after a differential leveling project to check if the inclination/declination stayed consistent, or if an inordinate amount of inclination/ declination exists. For Second-Order leveling or better, a peg test should be performed daily.

The following field procedures should be used to eliminate most errors during data collection:

1. Balancing sight distances eliminates most collimation and refraction errors.

2. Using two rods eliminates many systematic errors, if:

   a) BS and FS are read as close together time-wise as possible;
   b) Same rod is read first at each setup;
   c) Same rod is used to start and end each level run;
   d) A minimum ground clearance of at least 0.5 m is maintained.

Important checks are reading all three wires and meaning the three readings, and reading the rod inverted as the total length of the level rod is known. Precise leveling requires double running all lines forward and back (or use of the micrometer/invar staff method previously described) with a close comparison of closure between the two elevation differences determined for the same level run.

C) Topographic Data Collection

Topographic data collection differs from traversing for two major reasons:

1. The data has no mathematical quality checks since it is usually a series of sideshots.

2. The attribute (non-measurement) information collected is of critical importance in describing what feature is at a particular location.

Topographic data is usually collected with the total station by occupying a station of known horizontal (and possibly vertical) location. A station is backsighted in a known direction usually derived from the horizontal coordinates of the backsight. If the occupied station does not have a known elevation, a backsight to a point of known elevation must be made if elevations are desired (a backsight with 3-D coordinates resolves both the horizontal and vertical requirements). To derive an elevation from a backsight, a prism must be placed there so a slope distance and zenith angle can be measured. For horizontal direction, a prism is not required on a backsight but it is highly recommended as the measured horizontal distance can be compared to its value derived from coordinate inverse.

Since topographic data collection is often made from traverse stations (points of known coordinates), at least two other traverse stations can often also be sighted. This allows "double" backsights to be measured to each of the visible traverse stations, which provide added checks on the horizontal distance and elevation changes as measured. The horizontal angle between the two backsights, as compared to what is derived from coordinate inverse, can be checked by this method.

In some cases, a point of known location cannot be occupied to perform topographic data collection. Resection, a process of measuring to at least two known stations with angles and distances, or to three or more stations with angles only, is not recommended, as it is often susceptible to weak geometric solutions. Using this method, the elevation is derived by measuring a slope distance and zenith angle to any point of known elevation. As in the case of a conventional topographic setup, measuring to extra stations affords mathematical checks on the produced coordinates.

When the elevations of the topographic data are required, it is of utmost importance to measure height of instrument and height of target as precisely as possible.

Occupying a topographic feature with a prism pole is often impossible or impractical. A typical problem, locating a tree, is called an eccentric station since it is impossible to measure to its center. The eccentric station is located by using the prism to measure forward, back, left, or right of the object.

Two methods exist for eccentric measurement:

1. The offset distance (such as the tree radius) is measured and the slope distance, zenith circle reading and horizontal circle reading to the offset prism are recorded. The distance and direction (if a left or right offset) is corrected to the center of the object.

2. The slope distance, zenith circle reading, and horizontal circle reading are recorded to a left or right offset prism. The instrument is then pointed at the object for a second horizontal circle reading from which a horizontal angle can be derived from the prism to the object. Assuming a right angle at the prism enables the offset distance to be resolved and coordinates for the object established. This method requires two shots but does not require measurement of the offset distance.

Taping in orthogonal (forward, back, left, right) directions is another form of data collection for planimetric position only. It is effective for eliminating instrument setups from which only a few points can be measured and is normally recorded in conventional fieldbooks. An example is measuring to two corners of a building, which are visible from an instrument setup. The "back" corners of the building can be located relative to the other two corners by taping in orthogonal directions relative to them or any other surveyed points. Taping is simply a form of traversing where the tape replaces the EDM and all horizontal angles are 0°, 90°, 180° or 270°, or measured with a right angle prism or by human interpolation. Taping is not a precise form of locating, but it can dramatically speed up topographic data collection.

Another form of planimetric location by taping is location by station-offset. This procedure assumes known centerline stationing and all topographic features are located at a left or right offset distance perpendicular to a centerline station. The distance is simply taped perpendicular to the centerline.

If a level is set up on a centerline station and a station with known elevation is backsighted, using upper and lower stadia hairs for distance measurement and the middle cross hair for elevation determination allows 3-D station offset positions to be determined. The instrument person is responsible for

interpolating a right angle off the centerline for the level's line of sight in the offset measuring process.

Attributing data is critical in topographic surveys. One of the simplest attributes is whether the point measured to is a ground shot (for inclusion into a DTM) or a feature point, which is not to be included in a ground model, such as a measurement to a corner of a building overhang. Most of the points in a topographic survey are ground shots, while the number of feature points is quite few in comparison.

Another form of attributing involves assigning a station, offset, or cross section designation to a measured point. Examples are: a point is located at Station 82+18.00, a point is offset 12.6 m left of Station 82+18.00, or a point is part of the cross section data at Station 82+18.00.

Still another form of attribute information is a description of the measured object, often called a feature code. Feature codes are needed for both points (tree, manhole, property corner, etc.) and lines (edge of pavement, power line, building, etc.). MHD has a table of standard feature codes consisting of symbols, line types and colors for use on all Department projects. All field crews working on a project should be using the same feature coding system.

It is possible for a measured point to be both a point and part of a line, such as a power pole through which a power line runs. It is also possible for a point to be part of more than one line, such as when a point is the intersection of two centerlines.

Lines can be straight or curved (splines), and one unique line can have both straight and curved components. The measured points of a line receive the curve or straight designation. An incoming tangent, a horizontal curve and an outgoing tangent is defined by two straight points on the first tangent, a curve point anywhere on the horizontal curve and two straight points on the outgoing tangent. The line is then defined geometrically without having to locate the PC, PI, or PT of the curve. Lines which close upon themselves (buildings) should only require this designation, and not require a re-measurement to the initial building corner that was measured.

The final component of attributing is the descriptive information about points and lines not included in a feature code, e.g., tree type and diameter, manhole type and diameter or property corner description. Street name, driveway material or a utility line description are examples of descriptive information about line feature codes. The descriptive information is attached to the feature code and the point location. It can usually be reviewed

on the topographic map.

Whether using a data collector or a conventional fieldbook, drawing a field sketch of the topographic survey in a conventional fieldbook is of extreme importance for understanding the information later in an office environment. This is especially important for drainage data.

D) Digital Terrain Model (DTM) Data Collection

DTM-specific data is often collected as part of the topographic survey data collection process. All topographic features shot as ground points are included as spot elevations for building the DTM. In addition, other spot elevations need to be recorded to fill in the areas where topographic data did not exist. The density of spot elevation information required is a function of the job requirements and the terrain undulations.

A break line is a specific type of line feature code, which is required for most DTMs. Spot elevation information without any break lines in a DTM, will create a smooth surface with no abrupt changes in grade. Ground surfaces are rarely smooth in all areas as man-made improvements create many sharp forms of breaks (abrupt changes in grade) and some breaks are natural.

A "V" shaped ditch running alongside a road is an example of three break lines which will run fairly parallel with the road. The most obvious break line would be the bottom of the ditch. The two edges of the top of the ditch are also often break lines, as the change of grade perpendicular to the direction of the road is quite abrupt at these points. Top of curb and bottom of curb are also good examples of break lines. A line is surveyed topographically along the top (and bottom) of the curb. The points on the line can be designated as straight or curved as detailed in Section 3.2.2 (C), Topographic Data Collection.

Break lines are required to properly define a rock outcrop or other sharp changes in grade that occur naturally on many hills. The ground surface model will not be smoothed at the break lines and instead will show an abrupt change in grade. Profiles, contours, and 3-D views from the DTM should show these breaks.

E) Cross Section Data Collection

Before computerization made the concept of a DTM possible for ground modeling and volumetric computations, the development of cross sections was the standard procedure. A cross section is a profile of the ground surface at a given station perpendicular to the centerline. Any man-made features existing along the cross section line are generally shown on the cross section and labeled. Typical items as centerline, edge of lane, edge of pavement, curb, sidewalk, right-of-way line, etc. should be labeled on a cross section and thus must be surveyed and identified during the data collection process.

The distance between cross sections and the length of each cross section perpendicular to the centerline are dependent on the job requirements. Intermediate cross sections, not at a standard interval, are used to highlight dramatic changes in grade or elevation since cross section practice assumes a smooth transition between adjacent cross sections. Conventional cross sections also include subsurface structures, such as a manhole rim or catch basin, which need an elevation reference.

The first step for surveying cross sections is to identify the stations along the centerline. The cross section is then surveyed using a theodolite/EDM, a total station or a level.

Using a theodolite/EDM for cross sectioning is similar to topographic data collection. A station with known coordinates is occupied and a known backsight is used to derive direction and possibly elevation if the occupied station's elevation is unknown. Measurements are then made to selected cross section points, which are attributed as such and identified as to stationing. Software is used to compute the offset from the centerline to the measured point, as an office procedure. Software usually "moves" the measured position to a perpendicular to the centerline to be more graphically pleasing since that was the intended prism pole location.

Using a level, the centerline station could be occupied with a backsight to a point of known elevation. Foresights are taken to cross section points at that station with the distance left or right of centerline derived from reading upper and lower cross hairs. The level rod sometimes needs to be offset from the perpendicular to the centerline so that it may be read, but it is assumed that the determined elevation is the value on the perpendicular at that distance.

If it is not desired to occupy the centerline station, the distance of desired points left or right of centerline can be measured by taping. The level can then be setup anywhere and

measurements made to a rod at those points. The distance need not be derived from the level and rod as a tape was used to determine the point's offset relative to the centerline.

F) Stakeout

Stakeout is setting points in the field that were usually derived from office computations. Horizontal stakeout is a direct process where the monument, pin, or stake is set at the desired horizontal position. If elevation is required in the stakeout process, it implies a later earthwork and/or construction operation to establish a final defined elevation at that point. The survey point is placed at an elevation and referenced as to how far the final elevation is above or below the survey elevation.

Layout is usually from angle/distance using a total station or by station-offset from an existing centerline or baseline. The elevation component is derived from slope distance, zenith angle and height of target in the former case, and by level and level rod for the latter.

Layout with a total station requires coordinate inversing. If an occupied station, backsight station and desired layout station all have known coordinates it is possible to derive a horizontal angle and horizontal distance for layout. If the coordinates are state plane, the coordinate inverse distance will be grid and need to be divided by scale and elevation factor to obtain a ground value.

If the point cannot be laid out from the desired occupied station, another nearby station with known coordinates needs to be selected with an appropriate backsight and the coordinate inversing process repeated to obtain layout dimensions. A short spur traverse to set an intermediate point may be required to establish a position from which the layout can be performed.

Once the layout point is occupied, angles and distances to other visible stations can be checked. This is especially useful for important layout points like property corners.

Horizontal layout by station-offset is the use of a tape to locate the desired stationing along the baseline and then taping the offset distance at a right angle to the baseline. Scale factor and elevation factor need to be considered if the station-offset dimensions are in grid.

Vertical layout is establishing the survey point at an elevation by slope distance/zenith angle/height of target, or by level and level rod, and then noting the vertical dimension, above or below

the survey point, to the desired elevation. If the survey point is marked by a stake, the elevation difference and its direction is recorded directly on the stake.

Stakeout is identical in nature to "as built"-type quality checks where the survey is verifying the location of a structure relative to the design plans. Instead of staking where a position is, a surveyor is verifying that something is in the correct horizontal and vertical position. The as-built data checking is again based on coordinate inversing or station offset information.

3.3 FIELDBOOKS AND DIGITAL DATA COLLECTION

Historically, the surveyor's fieldbook has stored all of the numerical, graphical and descriptive data required to produce the desired final product. The surveyor's notebook also usually contains considerable information about the job, which may not affect the final product but is invaluable for survey indexing and validation purposes if questions arise regarding the survey.

The ability to collect survey data in digital form using a total station and data collector (electronic fieldbook, or EFB) has the potential for replacing much of the measurement information entered by hand in a conventional fieldbook. An EFB also usually has the capability of storing keyboard-entered attribute and other descriptive information, as well as non-total station survey measurement data such as taping, leveling (rod readings), station-offset, and eccentric distances.

If an EFB is being used but does not allow collection of desired attribute or descriptive information, this information must be recorded in a conventional fieldbook along with the EFB data and all field sketches. Use of a MHD Survey Notebook is required for all surveys conducted by or for the Department.

3.3.1 Capsule Description of Fieldbook Information Sections

Fieldbook data can be divided into the following sections:

a) Header - Descriptive information which contains the purpose of the job, its location, etc.

b) Calibration - Information on type of instrument(s), a test of its measuring ability, weather, field crew personnel, etc.

c) Setup - Required information about each instrument setup.

d) Observation - Required information for each measurement.

e) Linework - Usually applies to detail for topographic surveys where one is defining what a line represents (feature code) and which survey shots should be connected to one another.

f) Remarks - Generally descriptive comments which are not necessarily about specific survey points.

3.3.2 Detailed Description of Fieldbook Data Sections

A more detailed description of sections that can be used to provide needed fieldbook data is as follows:

A) Header

A header is information about a project. An example might be:

"This is Section B of a road widening project for Rte. 20 between Smallville and Pleasant Lake. This data is located approximately 8 km. east of Smallville and includes topographic data from Sta. 82+21 to 88+00. The existing two lane road is in a rural area, with unpaved shoulders and hilly terrain."

If an EFB is being used, the header should contain the name of the file as it was collected in the field.

B) Calibration

While some of this information could be included in a header, calibration information provides data on survey conditions, such as equipment used, field crew, and weather. Calibration relates to daily activities, while a header pertains to a segment of a project. Calibration information should include:

1. Comments - information about unusual field or equipment conditions
2. Temperature - degrees Celsius
3. Pressure - mm or mb of mercury
4. Weather - a description of conditions
5. Instrumentperson - name or initials
6. Notekeeper - name or initials
7. Rodperson(s) - name or initials
8. Instrument(s) - brand and model
9. Serial number(s) of instruments
10. Stadia factor - if relevant
11. Instrument calibration check(s)

Instrument checks for levels and theodolites/total stations are

described in Section 3.4, Conventional Field Data Processing. A level should have a peg test performed on it to determine any non-horizontal nature of the line of sight. A theodolite or total station should be pointed at a distinct point an equal number of times in the direct and reverse positions (recording horizontal and zenith circle readings) to determine the instrument's vertical and horizontal collimation errors and to obtain an estimate of any operator pointing error. An unusually large calibration error or change in value from the previous calibration error may be an indication of a problem with the mechanics of the instrument.

C) Instrument Setup

For differential leveling, data on instrument setups is recorded for necessary office processing. Measuring to a point of known elevation is required to determine the HI, the height (elevation) of the instrument's line of sight.

Components of a setup record for theodolite/EDMs or total stations should include:

1. Comments - Information about unusual field or equipment conditions.

2. Station name - Numeric or alphanumeric identification for a survey point, to be used every time the station is measured to or occupied; use facilitates association of coordinates with the station name.

3. Line attribute - Defines whether it is a curve or straight point on a line; not usually applicable to setup stations.

4. Feature code - May include further attributing such as a zone number or description.

5. Ground, feature, or cross section - Ground and cross section points are placed in a DTM while feature points are not. Cross section points are attached to a station.

6. Height of Instrument (HI) - A value not necessary in 2-D surveys, but needed in any 3-D survey performed with a theodolite/EDM or a total station. The meaning of the term has evolved from "elevation of the level" to "distance the instrument is above the mark".

Setup records should be consistent and easily interpretable.

In some EFBs the backsight is associated with the setup record. In other systems it is treated as an observation, which always

contains a horizontal circle reading, and may or may not contain a zenith circle reading and slope distance.

D) Observation/Measurement

Data will vary based on whether the data is collected by theodolite/EDM, total station, level (station-offset also possible), or orthogonal taping. Data consistent with all collection methods that should be recorded include:

* Comments - Information about unusual field or equipment conditions.

* Line attribute - Defines whether it is a curve or straight line; often not applicable to data collected with a level.

* Feature code - May include further attributing such as a zone number or description.

* Ground, feature, or cross section - Ground and cross section points are placed in a DTM while feature points are not. Cross section points are attached to a station.

* Sighted station name - Numeric or alphanumeric identifier for a survey point, to be used every time the station is measured to or occupied; use facilitates association of coordinates with the station name. A backsight station may be so designated, or it may be implied the first sighted station is the backsight. In some topographic surveys a point name for sideshots may not be required since only location and attribute data is desired.

1. Theodolite/EDM or total station data should also include:

* Horizontal circle reading - While this could be implied as zero on a backsight in some cases, it should be recorded to verify its value.

* Zenith circle reading - Not required if a prism is not at the sighted station or if a 2-D survey is being performed and the horizontal distance recorded is derived from the zenith angle and slope distance.

* Distance - Usually the slope distance, but may be a horizontal distance derived from the slope and zenith angle.

* Height of target - Not required if no distance is being measured to the point or if it is a 2-D survey; a necessity for any 3-D survey.

* Eccentric direction and value - Not required if the prism is over the survey point. If there is eccentricity, the direction (forward, back, left, right) of the prism relative to the object is required along with the horizontal distance from the prism to the survey point.

* Direct/Reverse "face" - If the zenith circle reading is not required but repetitions are being made, it is important to designate what face the instrument is in for proper reduction of the raw survey data. If the zenith circle is recorded, the instrument face is known.

* Position number - This is primarily reserved for control traversing where 1 direct and 1 reverse pointing represents a position. It can be used in other forms of surveying when one wants to reorient on a backsight station and then continue collecting other data at that occupied point. Toggling to the next position number requires measuring again to a backsight.

2. Data collected with a level should also include:

* Stationing of the measured point if horizontal location is desired.

* Offset of the measured point relative to the input stationing if horizontal location is desired.

* Rod readings - Should always include the middle cross hair reading and can include the upper and/or lower cross hair readings depending on the application.

* Route - A defined line, which contains points with stationing that make up the centerline to which the input stationing information is referenced. This allows coordinates to be computed on station-offset points if enough coordinates have been associated with some of the stations along the route.

3. Data collected by orthogonal taping should include:

* Occupied point name - Needed for coordinate computations.

* Backsighted point name - Also required for coordinate computations.

* Direction - Forward, back, left, or right relative to the occupied and backsighted stations.

* Distance - The measured horizontal distance from the occupied station to the station being measured to.

E) Linework

For topographic data collection, one of the most important pieces of information collected is how points are connected to form lines which define features such as centerline, edge of pavement, building, shrub outline, etc. A variety of procedures have been developed for use with both conventional fieldbooks and EFBs to define what points should be connected by lines. Many of these are based on collecting the points in order and having a change in feature code define a new line. There will be lines whose points cannot be collected in order and a system must be capable of the most generic case where a line is entirely defined by the names of the survey points which were collected. The linework is immediately tied to the points being identified as either straight or curved.

It is important to note that a point may have a feature code, while a line through it could also have a feature code (e.g., power pole and power line). Some points may not have point feature codes but will be part of a line that is feature coded (edge of pavement). Other points may not be part of lines (lamp post, manhole, etc.). A point may also not have any point feature code or be part of any line if it is a spot elevation and has properly been designated as a ground point.

Manually connecting coordinated points to form lines by computer in the office is a very tedious and error-prone process. Field survey collection procedures should facilitate the necessary office work to follow. The field note system must include a well defined procedure for line definition.

F) Remarks

Remarks are items of field information, which may help office processing of the survey data. It is common procedure to have a different person processing data in the office from the one who performed the field data collection. Remarks are useful in determining field procedures and finding mistakes. For example, a remark that the field person was not sure whether an occupied point was "A25" or "A26" might be resolved through an office check on the distance to a backsight to help confirm proper identification of the station.

3.3.3 Electronic Fieldbook (EFB) Data

The use of an EFB to collect and store raw data should follow the procedures previously described in this Section. Original raw data should always be preserved and any changes or corrections made to field data, such as station name, height of instrument or target, should be documented. Raw field data should also be preserved in hard copy output form similar to how a conventional fieldbook is preserved.

Procedures for proper office processing of data are described in Section 3.4, Conventional Field Data Processing.

3.3.4 Survey Notebook Data

Field survey data and sketches which cannot be efficiently recorded in the EFB should be recorded in a conventional fieldbook and stored with copies of the electronic data.

3.3.5 Notekeeping Examples

Appendix A-6, Sample Survey Notes, contains examples of various MHD notekeeping formats.

3.4 CONVENTIONAL FIELD DATA PROCESSING

This Section details some field procedures, which enhance office processing. It is important to use field techniques, which reduce the number of office computations. A field procedure, which identifies and corrects theodolite vertical collimation error, for example, will eliminate the need for later office correction and adjustment.

3.4.1 Procedures to Help Ensure Proper Processing of Field Data

Procedures to help ensure proper processing of field data include:

a) Use of field techniques and office procedures to eliminate or minimize all significant systematic errors.

b) Office processing procedures should take advantage of every redundant field measurement that can reasonably be made and should be geared to the reproducibility of final results (usually in coordinate form) from the raw survey data.

c) Proper analysis procedures should include the geodetic aspects of

conventional survey practices such as the relationship between geographic and grid angles and distances.

3.4.2 Removal of Systematic Errors

Most systematic errors are caused by incorrect or inconsistent field procedures or measurements. "Significant" systematic error is one which creates an error of more than: 0.01 m in any slope or horizontal distance derived from total station, EDM/theodolite or taping; 0.02 m in any trigonometry-derived elevation change; or 0.002 m in elevation change derived from differential leveling.

Horizontal angle systematic error, which causes an error in position, is a function of the length of the line and is derived from the distance multiplied by the sine of the systematic angular error. Trigonometric elevation difference error is derived from the distance multiplied by the sine of the error in zenith angle.

A) Taping

Systematic error in taping can be derived from the conventional equations for temperature, pull and sag. A better procedure is to make a measurement against a known baseline (usually 30 m in length). The surveyor can observe what the tape length is for a given temperature, sag and consistent operator pull. Those conditions can then be related to the survey field conditions and a correction applied, if necessary.

B) Electronic Distance Measurement (EDM)

Accurate EDM measurement requires a prism on the other end of a line, and thus any systematic error determined is actually a combined EDM/prism value. The most common systematic error in EDM measurement is incorrect prism offset. The prism offset should be known and recorded in the fieldbook or data collector.

Three procedures, based on measurement of horizontal lines, can be used to determine systematic error:

1. A tape is used to lay out a 30 m (or some other distance) baseline. The EDM measurement is compared to this baseline. The assumption is made that the taped baseline is more accurate than the EDM measurement.

2. Three collinear points (A, B, C, with B in the middle) of unknown distance are established. If EDM distances AB + BC does not equal AC then an error in the EDM/prism exists and can be calculated because the summed value contains twice that error while the direct measurement contains that value.

3. An EDM calibration baseline is utilized to determine any constant error from (1) or (2) above and also any existing scale error, i.e., different errors existing at different calibration distances. National Geodetic Survey's publication, NOAA Technical Memorandum NOS NGS-10, discusses the derivation of scale and constant errors from measurement of an EDM calibration range. Note that the procedures of (2) can be used on a baseline as the points are intended to be collinear.

In all three cases, all measurements need to be corrected for systematic errors of temperature and pressure before a correct analysis can be made. In the absence of a pressure determination, an estimate can be derived from the elevation of the survey area.

C) Zenith angle (vertical) collimation error:

The zero mark of a zenith circle of a theodolite or a total station is often not at the zenith when an instrument is leveled. The deviation of this zero mark from the zenith is obtained by precise pointings of a distinctly observable point in the direct and reverse positions. Repeated measurements ensure the validity of this calibration process. (See Section 3.2 - Field Operations with Conventional Survey Equipment). The direct zenith circle reading plus the reverse zenith circle reading should total 360°. The deviation from this value is twice the vertical collimation error. If direct and reverse values are measured as 80°00'20" and 280°00'10" respectively, the sum of 360°00'30" indicates that 15 seconds should be subtracted from all zenith circle readings. This correction is especially important on sideshots, as this error will not be eliminated since sideshots are generally only measured with direct readings.

The lengths of lines being measured and the magnitude of the vertical collimation error should be considered before corrections are made. Since most zenith angles are near horizontal, the chance of a vertical collimation error changing a horizontal distance by more than 0.01 m is highly unlikely. The elevation differences are generally affected more, but the accuracy of long lines in trigonometric leveling is rarely better than +/- 0.02 m. A large collimation error will also affect the computed horizontal distance as well as the elevation difference. If the instrument automatically uses the displayed vertical angle to compute the horizontal and vertical distance, it is most important to have the correct vertical angle displayed since D&R measurements will not correct for it.

Horizontal collimation error can also occur but is rarely of a magnitude worth correcting. To check horizontal collimation at the same time as vertical collimation, horizontal circle readings

should also be recorded. The direct reading should differ from the reverse reading by 180◦, with the difference being twice the horizontal collimation error. An error of greater than twice the Smallest Measuring Ability (SMA) of the instrument is significant.

Many total stations have internal compensators and software to apply corrections to the horizontal and zenith circles. Even after such corrections, a check should still be made for collimation error. Collimation values fluctuate and should be checked daily and recorded in a fieldbook or data collector.

D) Differential Leveling

The peg test is the standard procedure for determining the inclination/declination error in the instrument's line of sight. The simplest peg test involves determining the elevation difference between two points by a setup at the midpoint and a setup near one of the points. The midpoint-derived elevation is corrected for non-horizontal line of sight since the backsight and foresight distances are equal. The difference between it and the elevation determined from the endpoint is the inclination/declination error over the distance between the endpoints, usually referred to as inclination/declination per 100 m. The same value can be obtained by sighting two rods from different distances and comparing the derived elevation differences to obtain inclination or declination per 100 m. This value is known as the collimation ("C") factor. Any level with a "C" factor greater than 0.007 m/100 m should be adjusted. A "C" factor of less than 0.005 m/100 m is required For precise leveling.

3.4.3 Systematic Error From Refraction and Curvature

Differential leveling and zenith angles are affected by the systematic errors of atmospheric refraction and earth curvature. While the two errors are independent, both are generally considered together as one equation can effectively model the combined effect of both errors. The two errors affect elevation differences and zenith angles in opposite directions, but the atmospheric refraction error is only approximately 13% of the error for earth curvature. Earth curvature creates larger rod readings in differential leveling (a negative correction to the rod reading) and smaller zenith angles (a positive correction for direct position values). The equation for the correction to elevation differences is:

$$E_{C\&R} = 0.0675 \text{ X } D^2$$

where $E_{C\&R}$ is the error due to earth curvature and atmospheric

distance, and D is the distance, both in kilometers.

For extreme zenith angles (25° above or below horizontal) and measured distances greater than 1 km, a more complicated equation solves for the zenith angle error and also provides a small correction to the horizontal distance. Only in precise control surveys and an extreme zenith angle case should the more complicated equation ever be considered. Many total stations can apply an earth curvature and atmospheric refraction correction. The instrument manual usually contains the equation used.

Use of the equation for a variety of distances will show the correction to be a negligible value. It grows geometrically due to the squared term, but one should recognize the weakness of vertical accuracy in trigonometric leveling of long lines.

For differential leveling, the balancing of corresponding backsight and foresight distances eliminates the error due to curvature and lessens the effect of refraction. Refraction is a function also of the distance above the ground which is being sighted and is more pronounced for rod readings below 0.5 m. For differential leveling for control purposes, rod readings of less than 0.5 m should be avoided. Because of the squared term, balancing backsight and foresight distances for a level run does not necessarily eliminate the error due to earth curvature and atmospheric refraction. Due to the short foresight and backsight distances usually utilized in differential leveling and the standard practice of trying to balance backsight and foresight distances, earth curvature and atmospheric refraction correction can be considered negligible for all but very precise work.

A daily peg test should be included as part of control differential leveling procedures. The field notes for control differential leveling should always include three wire readings and a running total of backsight and foresight distances.

Control leveling performed by MHD is usually to Second-Order, Class I specifications, double run, using automatic levels with parallel plate micrometer and double-scale invar leveling staffs.

Most systematic errors in differential leveling can be canceled or at least minimized by following sound field practice. Some recommended procedures, which should be used, are:

a) Balancing backsights and foresights. Its value cannot be over stated since errors of off-level line of sight, curvature of the earth, and to a large extent, refraction are eliminated.

b) Using two rods, with BS and FS read as close together as possible. This reduces refraction and settlement errors.

c) Never reading a rod below 0.5 m because heat waves are more prevalent and refraction inconsistent near the earth surface.

d) Reading the same rod first at each set up, with the BS first on even numbered setups and the FS first on odd numbered setups.

e) Starting and ending a line on the same rod requires an even number of setups and reduces error caused by rod error.

3.4.4 Reduction of Repeated Raw Survey Measurements

Repeating survey measurements is standard practice for all traverse (non-sideshot) operations. It is possible to achieve suitable survey accuracies and closures without repeating traverse measurements. However, the advantages derived from repetition, the elimination of systematic errors through equal numbers of direct and reverse observations, and the possible increase in precision through averaging repeated measurements, far outweigh the short increase in observation time.

In addition to repetition, another recommended practice when using a total station or EDM is having a prism or reflector height measurement on all backsights. This enables checks on the horizontal distance and elevation change measured in opposite directions and on errors which repetition will not disclose such as in height of instrument (HI), height of target (HT) or reflector (HR), instrument setup over the point and reflector setup over the point.

If performing a series of sideshots from a control point backsighting another control point, a prism on the backsighted station can provide an excellent mathematical check by comparing the horizontal distance and elevation change to values derived from the control coordinates.

Backsight horizontal distance checks should generally be within 0.01 m + 20 ppm. A 1000 m distance should check within 0.01 + (1000 X 20/1000000) = 0.030 m. Elevation changes derived from trigonometric leveling should check within 0.02 m + 100 ppm. A comparison to existing coordinates, which may not be perfect, could contribute to the magnitude of the comparison.

The reduction of repetitions is generally a simple averaging process. As a by product of that process, standard deviations, standard deviations in the mean and the maximum spread of any single measurement from the average can be computed.

The maximum spread should be the major factor in assessing the quality of a series of repetitions. Horizontal distances should generally not have a deviation of more than 0.01 m + 10 ppm; trigonometric elevation difference maximum spread should not exceed 0.02 m + 50 ppm; differential rod readings which are repeated should not exceed twice the least count of the rod; in three-wire leveling the average of the upper and lower wires should not differ from the middle wire reading by more than two times the least count of the rod. The horizontal angle maximum spread should not be more than twice the Smallest Measuring Ability (SMA) on lines longer than 250 m. On shorter lines, the sine of maximum spread times the length of the shortest line (backsight or foresight) should not exceed 0.01 m.

A particular project may require specific standards for standard deviation in a single observation or standard error in the mean. The maximum spread from the mean remains the best indicator of the possibility of error in a repetition.

An obvious outlier should be discarded and not used in determining the mean. The most common standard for an acceptable angle is +/- 5" from the mean. When there is an unusually large spread between angles or if the angles appear to be creeping up or down in a consistent manner, the instrument and reflector setup data should be checked.

3.4.5 Reduction to a Datum

While some surveys may be based on assumed plane coordinates, most highway projects are of sufficient dimension that geodetic aspects must be considered. Computation of coordinates with respect to a datum may involve production of:

A) Geocentric Coordinates: an earth centered system within which GPS primarily works. If a combined GPS/traverse control network has been observed, this may be the most realistic system for coordinate production as the conventional measurements are also definable in this system. No application of scale or elevation

factors to horizontal distances are required in such a system. Software can readily convert values to state plane coordinates.

B) Latitude, Longitude and Elevation: the conventional geodetic coordinate system is well documented and applicable to their computation based on survey measurements. Unfortunately, the equations are not simple and are unavailable in most commercial survey reduction software. Horizontal distances must be reduced to the datum surface (ellipsoid) before coordinates are computed.

C) State Plane (SPC) or Universal Transverse Mercator (UTM) Coordinates: These are probably the most widely used geodetic reference systems for reduction of conventional survey traverse data, with state plane predominating for survey applications. Plane survey equations can be used if scale and elevation factors are applied to horizontal distances and convergence angles and LaPlace correction factors (a range from about 3.5" in the Boston area to about 10" in western Massachusetts) are applied to any astronomic azimuth determinations. The Massachusetts State Plane Coordinate System is a Lambert Conic Conformal Projection made up of two zones - Mainland and Island. Scale factors (the projection of a line from its position on the ellipsoid to the state plane projection) remain constant in an east-west direction and change most quickly in a north-south direction. The convergence angle changes according to relative position and changes predominately in an east-west direction.

It is critical to label distances as slope, horizontal, ellipsoidal/sea level, or grid. Likewise directions need to be defined as geodetic, astronomic, or grid when state plane or UTM coordinates and computations are being utilized.

3.4.6 Relationships Between Different Types of Distances, Angles and Azimuths

A) Distances

1. Mark to Mark - Spatial distance between stations. Can be obtained from slope distance by use of zenith angle or elevation difference, or by inversing geocentric coordinates (cartesian) obtained from GPS.

2. Slope - Distance measured from instrument to reflector.

3. Horizontal - Application of zenith angles or height difference to slope distance.

4. Ellipsoid (Sea Level) - Horizontal Distance X Elevation Scale Factor

Elevation Scale Factor:

$$\text{NAD 1927} = \frac{R}{R + \text{Elevation}}$$

$$\text{NAD 1983} = \frac{R}{R + (\text{Elev.} + \text{G.H.})}$$

R = Radius of Earth, approximately 6,372,200 m.

Elev. = Average Elevation of Line, in meters.

G.H. = Geoidal separation between Ellipsoid and Geoid (approximate Sea Level in Mass.), a negative value varying between 27 and 30 meters. This means that Ellipsoid heights are always approximately 30 m lower than orthometric or Sea Level height. With a G.H. in Mass. of between 27 and 30 m, using a value of 30 m for all calculations will produce an ellipsoid distance within 1 or 2 ppm.

The formula becomes:

$$\text{Ellipsoid Distance} = \frac{R}{R + (\text{Elev.} - 30)}$$

5. Grid = Ellipsoid Distance X Grid Scale Factor

The Grid Scale Factor is determined by the line's position in the state plane coordinate zone. In the Mass. zones, it is a function of latitude: 1.0 at the zone's two standard parallels and decreasing to 0.9999645 at the mid point of the zone. The factor is usually obtained from conversion software or from the table in Appendix A-5.

6. Uses of Distances:

a) Horizontal distances are used to define physical objects on the ground. Any topographic or detail survey or measurement which defines the true dimensions and location of an object should consist of horizontal distances.

b) A mark-to-mark distance is a true distance used with its earth-centered Cartesian coordinate differences in GPS surveys.

c) An ellipsoidal distance is used when working with geodetic

or geographic coordinates, i.e., latitude and longitude on the ellipsoid. The NAD 83 model is GRS 80.

d) A grid distance is used when working in the state plane coordinate system and reflects the geographic surface projected on a plane.

B) Angles

Angles as measured on the ground are geographic and no correction is needed for use with latitude and longitude coordinates. There is a correction known as the "second term" correction (or "t-T") to make measured angles suitable for use in SPC systems. This correction reduces the angle measured on the ground to a plane surface angle. Since the Mass. SPC system is a relatively small zone, this correction is insignificant for any location in the State and can be disregarded. In states with larger zones, the correction can be significant.

C) Azimuths

Azimuths are similar to bearings and are defined as the angle measured clockwise from the meridian. On the NAD 27 datum this angle was referenced from the south. On NAD 83, azimuths are referenced from the north so that "due east" is 90°, South 180°, and West 270°. The different types of azimuths and their relation to geodetic azimuths are:

1. Compass or Magnetic Azimuth - Needs to be corrected for magnetic declination. Declination in the Boston area is about 16° west of True North (Geographic North). A more precise value can be obtained from commercial software or from the National Geophysical Data Center, (303) 497-6478.

2. Astronomic Azimuth - Azimuths obtained from Polaris or Solar observation have to be modified by the "LaPlace Correction", (3.5" in the Boston area, to 10" in western Mass.). The proper value can be obtained from MHD, NGS, or NGS Deflect 90 and Deflect 93 programs.

3. Grid Azimuth - The correction for grid azimuth is the Convergence Angle. Its value depends on location in the state plane zone and is a function of longitude. In Mass. Mainland Zone, Grid Azimuth and Geodetic Azimuth are the same along 71°-30' West Longitude, the Prime Meridian. To the east of this line, the correction is defined as positive, which means the convergence is added to the grid to get the geodetic azimuth. To the west of this line, the convergence is defined

as negative and is subtracted from the grid to get the Geodetic Azimuth.

Convergence values are given in Appendix A-9, State Plane Coordinate System of 1983, or from coordinate conversion software.

Given a unique state plane coordinate pair in a defined datum and state plane zone, there exists one unique latitude and longitude. One unique latitude/longitude in a defined datum correlates to one unique state plane coordinate pair in the same datum and a defined state plane zone. When referring to coordinates related to a geodetic datum, there is usually no need to discriminate between geodetic coordinates and their state plane equivalents.

3.4.7 Horizontal Control Coordinates and Horizontal Datums

The North American Datum of 1927 (NAD 27) was based on the Clarke Spheroid of 1866 and was the basis for all positions (latitude/longitude) prior to about 1988. At the time of the definition of that datum, state plane zones were set up across the United States in English units.

In the 1980s the reprocessing of the data, which made up NAD 27, along with measurements made subsequent to that time, resulted in the North American Datum of 1983 (NAD 83). WGS 84 and GRS 80, two nearly identical ellipsoids that are associated with NAD 83, can be considered the same for all conventional survey applications. (See Appendix A-9, State Plane Coordinate System of 1983). This reprocessing (a least-squares analysis of the measurement data) produced "readjusted" latitudes and longitudes for all control stations. The shifts in positional information can amount to more than 100 m but are very systematic in magnitude and direction in a localized area. The geodetic direction between two points in a local area stays consistent (the change is normally less than one second) between datums. Unfortunately, an unlabelled latitude and longitude makes it difficult to tell which datum it belongs to, as the change in values between datums is at the level of seconds of arc.

State plane coordinate zones were redefined with different false eastings to make it easier to identify whether the coordinates belonged to NAD 27 or NAD 83, and all coordinates were published in meters instead of feet. A link to existing English units was critical and the U.S. Survey Foot conversion (1 m = 3.280833333333..... ft., or an exact conversion of 1 m = 39.37 in.) has been adopted by Massachusetts as the standard for conversion between units for all survey work. This conversion is required for all NAD 83 coordinates, which are in feet, bench mark elevations in feet, and any horizontal distances or elevation differences in feet.

Since NAD 27 is an English-units based system, all work should be based on the NAD 83 datum.

If NAD 83 equivalents are not available for NAD 27 coordinates, an accepted transformation procedure is NADCON, an NGS program that reads a data base of gridded estimates of coordinate shifts between NAD 27 and NAD 83 and computes weighted shifts for the desired station based on its relative location. The grid was developed from all control stations which had both NAD 27 and NAD 83 coordinates. Coordinates developed by NADCON should always be clearly identified as such.

NADCON only performs the computation in latitude and longitude. If a direct transformation from NAD 27 to NAD 83 state plane coordinates is desired, CORPSCON, a U.S. Army Corps of Engineers program, uses NADCON algorithms in the transformation process and in addition does all the conversions between state plane and latitude/longitude. Both NADCON and CORPSCON are public domain programs because they were developed by government agencies.

NAD 27 coordinates are generally of lower quality than those of NAD 83 because for NAD 83 much more data was available and the computations were performed on computer. It is preferable, therefore, to use control created in NAD 83 rather than NAD 83 control which has been transformed from NAD 27 coordinates.

Massachusetts is part of a New England High Accuracy Regional Network (HARN) based on NAD 83-92. This system was established after publication of NAD 83, so one may find local distortion between existing NAD 83 control and HARN control. If use of HARN is desired, control derived from HARN can be used and all coordinate information generated only from HARN, but the produced coordinates may not "fit" NAD 83 control coordinates in that local area. NAD 83-92 may become the official datum in time, but until it is densified sufficiently to make its use practical it will be used primarily on long range GPS projects.

3.4.8 Bench Mark Elevations and Vertical Datums

The first national definition of a vertical datum was the National Geodetic Vertical Datum of 1929 (NGVD 29), often referred to as "Mean Sea Level of 1929." Bench mark elevations were derived from elevation difference measurements primarily through differential leveling and were published in feet.

For reasons similar to those, which brought about the creation of a new horizontal datum, the North American Vertical Datum of 1988 (NAVD 88) was completed in 1991. A least-squares analysis was made of the vertical leveling control network throughout North America. Final bench mark elevations are published in meters and will probably differ from their NGVD 29 equivalents. The elevation change tends to be consistent in a localized area with variation in changes occurring as the area is expanded.

NGS has prepared VERTCON, a public domain conversion program similar to NADCON, except that it transforms bench mark elevations between NGVD 29 and NAVD 88 by providing an approximate latitude and longitude for the bench mark position. A conversion program is needed if bench marks in the old datum need to be converted to the new datum for a specific project. Version 2 of VERTCON, with refined values, is now available. VERTCON Version 2 is incorporated in CORPSCON Version 4.1.

It is allowable to have a project on NAD 83 and NGVD 29. In this case all bench mark elevations should be converted to meters by the U.S. Survey Foot conversion before computing metric elevations based on the survey measurements.

One should be aware that bench marks can often have systematic error tendencies in a local area. Large projects may require analysis and resolution of these systematic errors.

3.4.9 Reduction of Averaged Raw Survey Data

The first reduction of conventional slope distance/zenith angle/height of instrument/height of reflector values (averaged, if repetition is used) is to horizontal distance and elevation change. Except in precise surveys where correction for earth curvature and atmospheric refraction must occur, the horizontal distance is derived from the slope distance multiplied by the sine of the zenith angle; the elevation change is derived from the height of instrument, plus the slope distance multiplied by the cosine of the zenith angle, minus the reflector height.

The same horizontal distance or elevation change measured on multiple instrument setups (in same direction or in opposite

direction via a prism on a backsight) should be averaged. If the separate measurements are of varying quality (e.g., one with a prism mounted on a tribrach and one on a prism pole), a weighted average may be justified where the measurement of higher quality would have more effect in the averaging process.

Horizontal angles, which are repeated, should also be averaged based on the raw horizontal circle readings. If a series of traverse points are being observed, the backsight should generally be the longest line.

Any astronomic observations should be reduced by the standard hour/angle method. Repetitions are critical to assure the quality of this type of measurement. Care should be taken to avoid introducing a systematic error in the time or geodetic position used in the computations. The LaPlace Correction (about 3.5" in the Boston area to about 10" in western Massachusetts) conversion of astronomic to geodetic direction should be applied if significant.

3.4.10 Production of Horizontal Coordinates - State Plane Reductions

If work is in an assumed coordinate system, no factors will be applied to the values derived in the reduction described above. Since most transportation work is related to the state plane grid system, care must be taken to apply realistic scale and elevation factors to all horizontal distances, and convergence angles to any astronomic observations or geodetic azimuth mark values.

Elevation factors are computed by:

$$D_S = D_H \times [\, R / (R + h) \,]$$

where:

D_S is the horizontal distance reduced to the ellipsoid (often called sea level in this reduction), in meters;

D_H is the reduced "ground" horizontal distance in meters;

R is the approximate radius of the earth, 6,372,200 m; and

h is the ellipsoid height derived from the following:

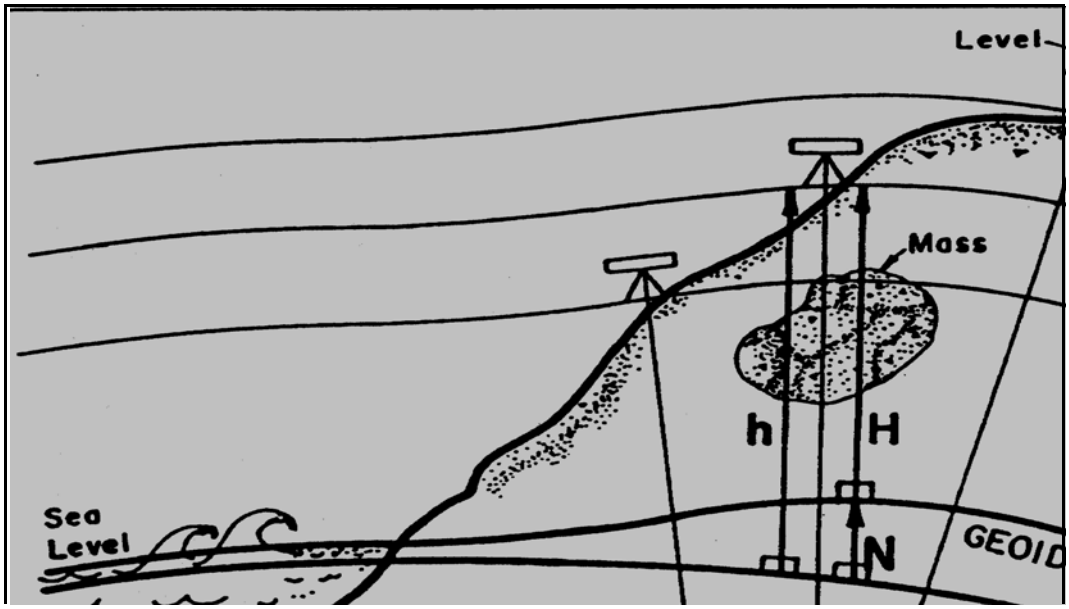
$$h = H + N$$

where:

N is the Geoid Height in meters (a negative number in Massachusetts), - 30 m can be used.

H is the orthometric height (commonly called the elevation), in meters.

While the ellipsoid is the reference surface for horizontal computations, the height of a point above it was rarely known prior to the introduction of GPS. Since the difference between an ellipsoid height and an elevation rarely exceeds 30 m, using elevation in this reduction causes a systematic error of approximately 1/200,000, which is negligible for most survey applications. To avoid the difficulty in obtaining ellipsoid height, elevation can be used.



One elevation "factor" can generally be used for an entire project. The elevation factor for the lowest and highest elevations of the project should be computed and any error introduced in using one project elevation factor should be verified as not significant. If it is significant, multiple or individual elevation factors must be used.

Scale factor is a function of the location of one's position in the zone and changes slowly in an area. Scale factor does not change in an east-west direction in Lambert zones like those in Massachusetts. Scale factors for the corners of a project should be computed and it should be verified whether use of one scale factor will not adversely affect the results. If the scale factor change is significant, multiple or individual scale factors should be used.

The most significant scale factor (difference from 1.000) in a Lambert zone is at the midpoint latitude of the zone. In the Massachusetts Mainland Zone this latitude essentially bisects the State and is thus close to most of the major metropolitan areas. The scale factor at this latitude is 0.9999645 and thus a 4 cm change occurs in 1000 m, which is well within surveying measuring abilities.

The use of software, which automatically calculates all scale factors, elevation factors and convergence angles, is highly recommended. Software, which requires user input of these values, creates the opportunity for transposition and misinterpretation errors.

Software is also available which computes all coordinates in a geodetic fashion although conversion of this to state plane coordinates will invariably be required. Sideshot distances must be reduced to grid prior to coordinate calculations. Grid distance (state plane distance) should not be used for physical distance, i.e. bridge spans, roadways.

3.4.11 Coordinates, Closures, Adjustments, etc.

Any redundant survey measurements produce some type of lack of fit due to random error, usually called closure. In complicated surveys there are many closures which can be calculated as one builds checks by closing traverse routes to, or through, known control points and bench marks, and by creating loop traverses. The least-squares approach is generally recognized as the most suitable adjustment of the random errors in a survey and it has effective reproducibility of results. This method is not required for any survey adjustments but is highly recommended.

Independent of the process used, the amount of adjustment placed on measured distances, angles, astronomic azimuths and elevation differences, generally termed a residual, should be looked at. If a measured horizontal distance reduced to grid is 500.052 m, and after adjustment is 500.073 m, the residual is 0.021 m.

Residuals should be within the random error measuring limits of your equipment and procedures. Generally, horizontal distance residuals should not exceed 0.02 m + 10 ppm, horizontal angle residuals should not exceed 1.5 times the least count of the instrument, except on short lines where the distance multiplied by the sine of the residual should not exceed 0.02 m, and trigonometry-derived elevation change residuals should not exceed 0.03 m + 20 ppm. Trigonometric leveling residual limits will be a function of the desired product, the equipment and procedures used, and the job requirements.

Care must be taken to ensure poor control does not systematically enlarge the magnitude of survey measurement residuals. Control values, which prevent job requirements from being met, must be identified and resolved.

3.4.12 Station-Offset Coordinate Computations

A standard field procedure often involves location by stationing along a centerline and measuring an offset left or right of that point at a right angle to the centerline. Coordinate computations are performed in the same fashion as any other calculations, with consideration for state plane reductions.

3.5 DIGITAL TOPOGRAPHIC MAP INFORMATION

Digital topographic map information is similar to conventional hard copy topographic map information and must meet MHD requirements:

a) Points - represented by a symbol (including color) at a defined scale;

b) Lines - any combination of straight and curved (spline), represented by line type, color and width;

c) Textual information - descriptive information about thepoints and lines, which cannot be represented by symbology.

The biggest difference is that the digital map is truly in ground units (meters), as the points and lines are made up of positions with ground coordinates. These positions can also contain elevation information. A hard copy map requires scaling or digitizing to obtain estimates of ground coordinates. Quality is a function of map quality and scale and the accuracy of the measurements made on the map. Elevation can only be derived from elevation representation, generally from contours or textual data such as spot elevations. Symbology in a digital map is defined in ground units by some map scale ratio to ground units.

3.5.1 Points

A point is represented by a symbol, which should be centered about the position that represents the surveyed point. The center of the symbol, or a mark in the symbol's center as shown on the digital map, should accurately locate the ground survey coordinates of the point.

Symbols are a series of lines (straight or curved) of defined color(s). A feature code (a numeric or alphanumeric label) is the identifier usually assigned in the field as the point data is being collected. Design and survey consultants should ensure that a copy of MHD's list of standard feature codes is obtained before work is commenced. A new feature code should be developed for unanticipated items that do not appear on MHD's standard list, but the MHD Boston Survey Office should be contacted as soon as possible to ensure the coding is satisfactory.

3.5.2 Lines

Lines (often called strings or chains) are a series of points, which are required because the planimetric feature cannot be defined well enough by a simple point. Centerlines, building edges, edge of pavement and shrub outline are all planimetric features which require lines between multiple surveyed points to enable a correct map representation. The points in a line can be defined as curve (spline) or straight. Different software defines different types of splines and how these interact with straight sections of lines.

A centerline is an example of a line with possible straight and curved sections. To define an incoming tangent, circular curve and outgoing tangent, one needs two points (straight) on each tangent and one point on the curve (curve). Software can resolve the entire horizontal curve from this geometry without field location of the PC, PI, or PT.

A building is usually all straight lines. A shrub outline would usually be defined by curves representing the planimetric outline of the feature.

Each line feature code would be represented by a line type and color. Line type might include continuous, dashed, dots, symbols instead of dots, etc. A combination of these types might be used to expand the types of lines where many feature codes are required. Color might be used to distinguish between two related lines types such as centerline and edges of pavement. Line width may also be a definable component of line feature coding.

For a line that closes back upon itself, such as a building, it is desirable to have the same exact coordinates on the end points of the line. Having to re-shoot the initial point as the last point will result in slightly different coordinates and thus not a truly closed figure.

Points in a line may also require point symbology. An example is power poles requiring point symbols but also defining a powerline.

3.5.3 Descriptive Information

Descriptive information about a point or line is often required, as the symbology itself cannot express all of the necessary information. A feature code for tree may require a description of type, diameter, and crown. A manhole may require type, size, etc. A centerline may have a street name as a description. The surveyor must be aware of what descriptive information is required for planimetric features for a particular job before starting the data collection process.

Descriptive information is usually shown at a defined height and offset in a standard fashion from the point or line. In areas where many points are described, a computer screen can become crowded with descriptions. Having software, which shows descriptions for only certain feature codes at one time, can be an extremely useful function.

3.5.4 Other Features of Topographic Information

A break line for Triangulated Irregular Network (TIN) models and production of DTMs could be a special type of feature code or a special on-off attribute associated with all lines. Similarly, points need to be defined for DTM purposes (spot elevations) or simply as a feature, which is not part of a ground model. An on-off "toggle" could designate whether points and lines are part of the "ground" model or that they are "features" whose elevations should not be included in the DTM. Additional attributes (such as cross section point) could also be attached to a point.

Most lines and points are part of the ground model. Feature-only points and lines are usually a small subset of the amount of data, which has been collected.

Another useful way to segregate data is through use of zones or levels. Zones could be by elevation (sub-surface, ground, elevated) or by groups of feature codes (utilities, roads, hypsography, etc.). Some software systems enable the on/off toggling of certain zones or layers of information so a user may concentrate on a smaller amount of data.

3.6 DIGITAL TERRAIN MODEL AND CROSS SECTION DIGITAL DATA

While the conventional cross section approach to roadway design problems related to a ground surface has been a standard for many years, advances in both computer hardware and software have greatly increased the use of Digital Terrain Modeling.

A Digital Terrain Model (DTM) is a ground surface estimator, which is generated from survey data collected on the actual ground surface. The surface data is comprised of spot elevations (with known planimetric location) and "break lines" (known X,Y,Z of points on the lines) which define abrupt changes in the grade of the ground surface. A break line runs along the abrupt change; top of curb, bottom of curb and bottom of ditch are all examples of break lines. An advantage to data collection via DTM is that data does not have to be collected in a specific gridded or cross section fashion. Occasionally, shots taken in cross section fashion are a useful adjunct to DTM spot elevations and break lines.

3.6.1 Development of a Digital Terrain Model (DTM)

A data set of spot elevations and no break lines would imply no abrupt changes in the grade of the ground surface and the following simple example explains how DTMs are generated. Following that is an explanation of how break lines fit into the development of the DTM.

A DTM is generated by initially creating a Triangulated Irregular Network (TIN) of the spot elevations. A TIN is simply a series of interconnected triangles between the spot elevations. No lines cross as the nodes of the triangles are the spot elevation locations. TIN software tries to create the smallest and most equilateral triangles from the spot elevation information. The biggest deviation from these types of triangles will usually be at the edges of the surveyed area.

Since the TIN lines do not cross each other, three survey points, which comprise a triangle in the TIN, have no lines inside of that triangle. The edges of the triangle represent a line along which the DTM would assume a constant grade between the survey points (linear change in elevation). A contour, which enters a triangle based on elevations of the nodes, has its location linearly interpolated between the node points. If a contour line passes through one edge of the triangle it must pass through one and only one other edge of that triangle. A contour line can thus be generated from a TIN by interpolating its location from edge to edge of interconnected triangles in the TIN. Software can smooth the curve of the contour to a user-defined level.

To get an estimate of the elevation of a point inside a triangle, a weighted average (the weight is inversely related to the distance from the point to a triangle node) is used to calculate it from the elevations of the nodes of the triangle that it is contained in.

A profile can be derived from a TIN by defining the desired line and then having software find all of the intersections of that line with lines in the TIN. At each of these intersections an elevation is interpolated between the ends of the triangle edge and from this information the profile can be drawn. Cross sections are drawn similarly but instead are at right angles to the defined line at a defined interval and at a defined extent left and right of the defined line. Centerline and edge of pavement profiles are then derivable from a TIN along with any form of cross section information.

The value of the DTM aspect of the TIN procedure for representing 3-D data is in determining the volume between a design surface and the actual ground. Instead of the historical process of deriving volumes between cross sections, DTM provides a comparison of the two 3-D surfaces. The surfaces are broken up into very small grids and the volume between the two surfaces of each of these grids is computed and totaled. The small trapezoidal areas from the small grids are equivalent to a calculus integration relating the two surfaces.

To incorporate break lines into a DTM, software creates a series of small triangles immediately to the left and right of the break line. These small triangles cause any profile running perpendicular to the break line to show a distinct change in grade at the break line. Break lines allow even vertical features such as top and bottom of curb, or a vertical wall, to be represented in the DTM, allowing more precise volume calculations to be made in the area of the break in grade.

Break lines enhance 3-D perspective viewing of the data. One can envision a computerized view looking down a road and see the top and bottom of curb and a ditch correctly represented. The derivation of a spot elevation from the TIN in areas of the break line is well defined by the inclusion of break line data into the DTM modeling process.

3.7 GLOBAL POSITIONING SYSTEM (GPS) USE IN SURVEY OPERATIONS

GPS is a very useful tool for the collection of high-accuracy positional survey data. This discussion assumes that centimeter level accuracy is desired and that a check on all positions will be made using redundant measurements. Lower accuracy requirements or non-redundant forms of GPS surveying are to be utilized only if appropriate for job requirements. GPS uses for navigation and other

comparatively low-accuracy positioning applications are not included in this discussion.

Field procedures described below concern conventional static differential GPS surveys. Other techniques such as rapid static, pseudo-static, pseudo-kinematic, or kinematic may be applicable to certain job requirements.

A more in-depth discussion of GPS uses and Alfred Leick, 1990, Wiley & Sons, Inc, can find procedures, and a list of other sources, in GPS Satellite Surveying.

3.7.1 Global Positioning System (GPS) Field Equipment

Receivers must be capable of centimeter-level accuracy when used in differential mode and the information is processed to a baseline vector.

A) Single vs. Dual Frequency Receivers

On lines less than 20 km in length, single frequency receivers may be used. Dual frequency (L1/L2) receivers may be used but on lines of less than 10 km, processing of the data should be in single frequency mode. When measuring lines longer than 10 km with single frequency receivers, one should occupy points in a session for at least 1.5 hours to enable better ambiguity resolution.

B) Antenna Mounting Systems

The antenna for a GPS receiver should be mounted on a tribrach secured to a standard surveying tripod. The tribrach should have an optical plummet (preferably rotatable) with a cross hair for centering over points that are to be positioned. Tribrachs should be checked at least bi-monthly for proper adjustment by methods such as a tribrach circle, placing two securable tribrachs on top of each other, or checking centering over a point with a plumb bob.

When initially securing a GPS antenna/tribrach combination, one should have assurance from the manufacturer that the center of the GPS antenna and the cross hair of the optical plummet of the tribrach are vertically aligned.

C) Height-of-Instrument Measurement

A mechanism must be provided to precisely measure the height of antenna over the point to 0.001 m without interpolation. A graduated metal alloy (or similar) device provided by the GPS

antenna manufacturer should be used for this purpose. The GPS receiver should provide the ability to measure a non-vertical diagonal value to a mark(s), for a check on the antenna with correction by office processing to a vertical component. Because of the importance of a correct height-of-instrument determination, an excellent check is to make the measurement in both feet and meters with two different measuring devices.

In conventional surveying, height-of-instrument and height-of-reflector measurement is unimportant if elevations are not required. This is not true for GPS surveys. Even if only 2-D positions are required, height-of-instrument measurement is critical because GPS measures in a geocentric (earth centered) coordinate system, which is not gravity, based. Any offset from the antenna to the point becomes critical, as it is a shift in all three geocentric coordinates.

3.7.2 GPS Field Procedures

A) Mission Planning

Mission planning before a project is critical to ensure the quality of the output data. Each station should be visited so that the satellite visibility window can be estimated. The extent of any azimuth or elevation angle blockage to the satellite window should be documented. Where obstructions exist, individual site information must be entered so the mission planning software can omit from the solution any satellite information from behind the obstructions.

B) Geometric Accuracy

Accuracy of the data being collected is a function of the number of satellites and the geometry of the satellites relative to the receivers. A variety of accuracy indicators collectively referred to as Dilution Of Precision (DOP) is generated in GPS surveying. Positional DOP (PDOP) is usually the best overall indicator of GPS data quality. Best quality is identified by a low PDOP, and conversely a high PDOP indicates a weak solution due to low number of satellites visible or poor satellite geometry. Data collected when the PDOP is higher than 5.0 is generally considered non-suitable for survey accuracy requirements. If the poor PDOP only exists for 5-10 minutes in a longer (1+ hour) occupation time it will generally not produce adverse results. If poor PDOP data does appear to be contaminating the results, it should be eliminated and reprocessed. If poor results still exist, reoccupation of the line is required. At no time should data be used where less than

four satellites are available. PDOP should be evaluated during mission planning and monitored during all field data collection.

C) Elevation Mask

An elevation mask of no less than 15 degrees will be used in all mission planning, data collection and data processing. Satellites within 10 degrees of the horizon generally provide poor geometric solutions, and that data can adversely affect the results from satellites, which are above the mask if incorrectly included in office processing. If office processing is showing questionable results, one may consider masking out data below 20-25 degrees and reprocessing.

D) Occupation Duration

Occupation times for stations should follow FGCC GPS control standards for the level of accuracy required for the project (See Appendix A-11; also available from NGS). As an example, one class and order of survey requiring 60 minute session occupation times for all stations is designed to achieve 20 parts per million or 1/50,000 closures. Even if a 50 minute occupation time yields better accuracy, it is not considered suitable, i.e., field requirements must be met independent of accuracy requirements. Occupation times can vary, but for a particular session the common observation time of all receivers must exceed the minimum value required for the job.

E) Data Collection Interval

GPS data should be collected at no longer than 20 second epochs (time interval between when signals are received). While faster collection rates should not cause less accurate results, neither will faster rates significantly improve results in conventional static mode. Short occupation times may warrant faster collection rates though this practice limits geometry considerations as the satellites will move a shorter distance. Storage requirements and longer data processing times also detract from the use of faster collection rates.

Data collection at Continuously Operated Reference Stations (CORS) is at 30 second epochs. Collection epochs can vary depending on the type of GPS survey.

F) Session Planning

The health of each GPS satellite should be checked when planning a GPS survey, and that status should be monitored during the survey. Unhealthy satellites do not provide useable data and their use should be avoided.

Sessions for a day should be labeled numerically (1, 2, 3, etc.) or alphabetically (A, B, C, etc.) as a standard component of mission planning and station occupation procedures.

Careful planning of station occupation is a critical step prior to GPS field work. Every party member should have a list of the order of station occupations and the times during which those stations will be occupied. If one receiver is collecting data when the other receivers are not, that data is not useable. Likewise, arriving at a station 10 minutes late may cause that station's occupation time to be less than the job requirements. Each station should be assigned a unique four character name/number and that station designation should be included in the document describing the field sessions.

G) Station Occupation Requirements

In occupying a station in the field, the surveyor should first confirm that the correct station has been found by verifying an existing field sketch and ties to objects. A sketch of the point and its surrounding topographic features should be made in a bound fieldbook. Any disturbance of the monument should be documented and if possible a fieldbook rubbing of the monument should be taken. Extreme care should be made to secure tripod legs and to ensure that the GPS antenna is secure in the tribrach and precisely centered over the station. The receiver ID number, height of instrument and temperature and barometric pressure should be measured and always recorded in a fieldbook and also into the field computer, if possible, before and after the session. The 4-character station name and the session label should be entered into the fieldbook and, if possible, into the field computer. The fieldbook should also contain the session start and stop times and any other information, which will ensure accurate and efficient office processing.

Any malfunctioning satellites should be omitted from the data collection process. If a surveyor does not become aware that a satellite is malfunctioning until after returning to the office, the satellite must then be tagged as omitted.

H) Building Checks into the GPS Network

The reoccupation of a station in the next session does not mean simply leaving the instrument setup exactly as the previous session. The tripod must be reset and the antenna repositioned exactly over the station. This will require re-measurement of the height of instrument, providing a check on that value in both sessions, and a check on the centering of the antenna over the station. The second session should have the same fieldbook and computer information reentered.

In control densification projects every station in the project must be occupied in at least two sessions. For most projects, if a station is only occupied for two sessions, those two sessions cannot contain the same combination of other stations. This means if Stations 0001, 0002 and 0003 are occupied during Session A, occupation of the same stations again during Session B does not constitute two occupations of those stations.

An exception to the previous rule is made for a small project such as the establishment of control for a bridge survey. In such a case, two control points will be positioned in proximity to the bridge from two existing control points. With four receivers, two sessions on the same points is a reasonable approach. Between the two sessions each setup should be taken down and reestablished as a check on the height of instrument and the positioning of the antenna over the point. Fewer than four receivers could be used, but more sessions and some movement of receivers would be required.

3.7.3 GPS Office Processing Procedures

Traditional differential GPS baseline processing should be used in the conversion of raw GPS data to GPS vectors (measured difference in geocentric coordinates between two stations).

A) Checking Field-Entered Data

Before beginning the mathematical processing for a session, all station names, heights of instruments, temperatures and barometric pressures should be verified as correctly entered. If the data was not entered in the field, it should be entered before processing. Data found to be incorrectly entered by checking fieldbooks should be corrected before processing is begun. If it is known that certain satellites were not operating properly (health of satellites is available through electronic bulletin board services), they should be tagged and omitted from baseline processing if this was not already done in the field.

B) Baseline Processing

There are three types of standard baseline processing usually performed: double difference fixed solution, double difference float solution, and triple difference solution. Since the processing algorithms are slightly different, each solution produces a slightly different value for the vector. Noisy GPS data, short occupation times, poor satellite geometry or long measured lines can preclude generation of a fixed solution. If a fixed solution is obtained, it is recognized as the "best" of the three solutions and should be utilized. On lines of less than 10 km in length, the inability to produce a fixed solution often indicates the vector produced by the other methods is of suspect quality. Measuring lines longer than 20 km in length with single frequency receivers is not recommended because it often fails to produce fixed solutions. Dual frequency data, processed using L1/L2 frequencies, will be more successful at producing fixed solutions on long lines. Use of a float or triple difference solution in absence of a fixed solution is also not recommended. The correction for failure to reach a fixed solution is reoccupation of the line.

C) Evaluating Vector Processing Quality

Quality control indicators in baseline processing output are of three types:

1. Error estimate of the vector:

   A "plus or minus" in the quality of the DX, DY, DZ (change in geocentric X, Y, Z) between the two endpoints of the vector is estimated from the baseline processing results. It is derived from how the individual data epochs fit one another. The error estimates are in meters and need to be considered relative to the length of the line: an error estimate DX of 0.100 m on a 100 m line would not generally be considered acceptable as 0.1/100 = 1/1000 for a relative error estimate. By comparison, an error of 0.1 m on a 10 km line, or 1/100,000, is typical of standard GPS control surveying.

   If 1/50,000 results are desired, an error estimate, which produces a relative error of 1/20,000, should be considered suspect, but it should not be considered unsuitable until the "fit" with other measured vectors through loop closures and least-squares analysis are checked.

   The root-mean-square (rms) error estimate of the vector is a cumulative error estimate derived from the DX, DY, DZ error estimate values, and can be used in lieu of the individual error estimates of DX, DY, DZ.

2. Number of epochs used vs. total number of epochs:

   This reflects the number of satellite epochs (measurements), which were considered by the software as realistically "fitting" the other epochs. Experience has shown that if less than 95% of the total number of epochs was used (5% or more were rejected) then the vector may be of unsuitable quality. As in (1) above, it should only be considered as suspect at this point.

3. Ratio error:

   This is a quick indicator of the strength of the solution. The optimal solution results in 100.0 for a ratio indicative of excellent resolve of solution ambiguities. The optimal ratio will normally be obtained under good satellite geometry and sufficient occupation time. Ratio errors of less than 98 should be considered unsuitable. Any ratio less than 100 should be considered suspect and closely reviewed in subsequent network analysis.

   The true test of the quality of the GPS results is not achieved by baseline processing. Incorrect instrument height, failure to center over a point, incorrect control coordinates and misidentified stations are examples of errors not resolved at this point. The next step is examining the internal quality of all GPS vector information and the integration of all sessions' vectors.

3.7.4 Non-Trivial vs. Trivial GPS Vectors

Before data quality can be examined, the principal measurements produced from baseline processing must be defined. One session with two GPS receivers will produce one vector measurement between the two stations occupied. The DX, DY, DZ values from station A to B are equivalent but opposite in sign to the values from B to A. In this case A to B and B to A are not two measurements. A reoccupation of A and B in a subsequent session would produce a second vector measurement between these stations to compare to the value obtained from the first session.

If three GPS receivers are being used per session, baseline processing will produce three vectors. For a given session, the third vector is always the product of the first two. The DX, DY, DZ values add up to zero, minus round off. For three receivers in a session, two non-trivial and one trivial vector is produced; only two vectors have been measured, and a third vector is generated from the other two.

Having "n" GPS receivers in a session produces "n-1" non-trivial vectors, most easily visualized as all vectors in a session being from one of the stations to the other "n-1" stations. What station is selected as the "from" station for a session should be defined during mission planning to ensure at least two occupations of all stations. Baseline processing of all combinations (trivial and non-trivial) vectors does no harm, but does require longer computation time. It is important to realize a loop closure using all vectors in a session will produce perfect results minus round off because of the nature of the computations. Likewise, using all six vectors (3 non-trivial, 3 trivial) in a four receiver session as part of a network analysis provides misleading results as the network will appear to have more redundant checks than it truly does. The solution is to never use trivial vectors in any analysis of GPS solutions. The rest of this discussion refers to analysis of only the non-trivial vectors.

3.7.5 GPS Loop Closures

The simplest procedure for evaluating network results is a loop closure with the sum of the DX, DY, DZ components in the loop. Due to the random errors in the measuring system, a misclosure in DX, DY, DZ will be produced in a loop. The square root of the sum of the squares of the misclosures in DX, DY, DZ is the linear misclosure. The sum of the vector lengths is the total distance covered, and thus the linear error of closure divided by the total distance in the loop produces the traditional ratio error of closure. While in conventional surveying this value is usually in 1/x form, in GPS this value is usually given as a ppm (10 ppm = 1/100,000). Any loop closure should yield a ratio closure or ppm better than the accuracy requirements of the job.
Every vector should be included on at least one loop closure computation. If a closure indicates an error, performing different loop closures can help identify the vector which is the most logical source of the problem.

Propagating the error estimates in the vectors along the loop enables computation of the estimate of the closure. This will be a one standard deviation (one sigma) estimate, and form a normal distribution (bell shaped) curve. The closure can only be expected to be better than the error estimate 67% of the time. This is why the outer limit for a sure method of error detection is considered three times the propagated value; 3 sigma is approximately 98.6% confidence.

In the case of small loop distances, the probability of obtaining a very small ppm closure is limited by the short distance. Closure for a small loop of better than 0.01 m can be considered suitable for most control work, independent of the resulting ppm.

3.7.6 Least-Squares Analysis and Error Estimation

Least squares is the most accepted process for analyzing a GPS network. The limit of the loop closure is that it can only look at a portion of the data. Least squares uses the error estimates of the measurements in evaluating where misclosures on vectors, called residuals, should exist. For example, a vector with a DX error estimate of 0.1 m will generally have a larger residual than a vector with a DX error estimate of 0.01 m. It is possible that the first DX may have a smaller residual simply because it fits the rest of the data better. A residual can thus be considered the difference between the measured and the adjusted quantity based on the produced final coordinates.

Error estimates for vectors being subjected to the least squares are usually generated by one of the following methods:

a) Error estimate results from baseline processing are used, or

b) The error estimates are similar to electronic distance measurement errors, which are usually a constant plus a ppm. A line receives an error estimate no matter how short it is (constant error) and the error estimate grows for longer measured lines (ppm error).

Each method of error estimation is considered valid, each is suitable for least-squares analysis and each is used by commercially available software. MHD recommends method (a) above as those error estimates are related to the conditions that existed at the time of the data collection, e.g. noise, atmosphere, window blockage, cycle slips, etc.

3.7.7 Minimally Constrained Least-Squares Analysis

The first goal in network analysis (as opposed to individual loop analysis) is verification of the internal fit of the GPS vectors among themselves. In this analysis the final coordinates are not important and the standard procedure is to hold one of the stations fixed in 3-D position based on its approximate coordinate values from single point positioning data generated during vector processing. Holding only one point fixed (minimally constrained) ensures the residuals of the GPS vectors will not be influenced by control coordinates. Mathematically there is no check on control coordinates in a network with only one fixed station and likewise any error in control coordinates will not be propagated into the residuals in the GPS vectors.

The minimally constrained adjustment is an indicator of the quality of the GPS independent of the control which was occupied and is analogous to a conventional traverse with one known horizontal station, one bench mark and one fixed bearing. The value of residuals in DX, DY, DZ components should be close to their error estimates and if it is more than three times the error estimate the measurement is considered suspect. A ratio error of closure can be calculated for the vector by dividing the square root of the sum of the squares of the DX, DY, DZ residuals by the length of the line and that can be converted to a ppm equivalent. No 1/x or ppm derived from the residuals should exceed the tolerances for the job based on the survey accuracy required. The exclusion is short lines in which independent of 1/x, a linear residual of less than 0.01 m is acceptable for most survey applications.

3.7.8 Constrained Least-Squares Analysis

When suitable results from the minimally constrained analysis are confirmed, the next step is to see how the GPS vectors fit the existing control stations, which are part of the network. One difficulty in GPS surveying is that the measurements can often be of better quality than the control coordinates. Holding less precise control coordinates fixed will result in GPS vectors with larger residuals than reflected by the minimally constrained analysis.

A) Control Issues

Control can be 3-D, 2-D or 1-D in nature. While in route projects the 3rd dimension is elevation relative to the Geoid, often approximated and referred to as mean sea level, GPS does not directly measure elevation differences. The third dimension in GPS is distance up or down from the ellipsoid (Clarke 1866 in NAD 27 and GRS 80 in NAD 83).

B) Relation of the Ellipsoid to the Geoid

In a local area (no larger than 10 km in any direction) where no gravity anomalies are present, an ellipsoid height difference and an elevation difference between two stations will be within 10 cm. The direction of gravity and a perpendicular to the ellipsoid will also be near coincident and the ellipsoid height difference between two stations can be assumed to be very near the elevation difference in magnitude for GPS networks in a local area. This assumption is valid for surveys where precise elevations are not required and is within the accuracy limits of trigonometric leveling. The exact difference between ellipsoid and orthometric heights can be determined by comparing ellipsoid height difference obtained by GPS with orthometric height difference obtained by leveling.

In a local area, the elevations can be considered ellipsoid heights in what could be called a local ellipsoid solution. In a least-squares adjustment to determine 3-D position, the correct elevation should always be the ellipsoid height. To test the reliability of elevations being produced, withhold some of the bench mark elevations as control and see how close the least-squares results are to their published values.

The solution to resolving the ellipsoid height to elevation issue is to occupy three or more bench marks in every GPS network. At least three horizontal control points should also be included in every GPS network, because if one of the GPS points is in error, having only two control points can identify the problem but not resolve it. While lack of window due to obstructions or time constraints may prevent some control points from being occupied, preference is that all control of suitable quality in an area should be part of a GPS survey to ensure its consistency.

It is highly recommended that all GPS surveys be tied into the HARN (High Accuracy Regional Network). The HARN is the basis for the NAD 83-92 adjustment. At this time, NGS is publishing only the NAD 83-92 values for its horizontal positions. Another important reason for using HARN is that it provides an accurate ellipsoid height. To obtain good orthometric heights for a GPS network, several things are required:

1. At least one, and preferably more, accurate ellipsoid heights;

2. At least three accurate elevations of GPS points;

3. A method of determining Geoidal separation at all points within the network.

With the introduction of Geoid 96, determination of orthometric heights with GPS has become easier and much more accurate. Geoid 96 has an accuracy of +/- 0.01 m and its use in a network with known ellipsoid and orthometric heights allows the determination of elevations to within +/- 0.01 m. Unless stated otherwise by MHD, differential leveling will produce all control elevations.

C) Biased Constraints

The constrained least-squares analysis normally allows "biased constraints" to be defined, which allows rotation of the GPS vectors to better fix the control points in the area. This is especially useful when using elevations directly because the Geoid/ellipsoid separation changes slowly over an area, which amounts to a rotation. The biased constraints should be used with extreme caution as "rotating" the GPS survey to fit unreliable control coordinates is an incorrect process. The unreliable coordinates should be identified and not used.

D) Control Evaluation Process

The most logical way to start the constrained least-squares evaluation process is to add control in sequentially. Start with the best horizontal and vertical control station(s). One 2-D and one 1-D as control is the equivalent of a minimally constrained analysis. Run the least squares and examine the "closeness" of the produced coordinates of control, which was withheld to the published values. If the results appear in close agreement, those control coordinates can probably be added to the adjustment without adverse effect. Control coordinates that don't match well should be added last or not at all due to lack of fit with the other control coordinates in the network.

As you add more control the least-squares analysis, statistics may worsen, due to imperfections in control coordinates. If the data becomes dramatically worse, a suspect control coordinate has been entered and must be eliminated or otherwise resolved (e.g., the coordinate may have been entered in the computer incorrectly).

E) Identifying a Realistic Analysis Result

A realistic constrained analysis will produce vector residuals, which are no worse than the job accuracy requirement. If desired accuracy is 1/50,000 (20 ppm) no vector should have a residual greater than 20 ppm except for extremely short lines where the 0.01 m residual can be used. If the minimally constrained analysis produces reliable results, and if with all control entered the constrained analysis shows results, which do not meet, desired accuracy, the surveyor must determine which control points are suspect and they should be removed from the constrained analysis.

All control coordinates must be in the same datum. HARN network coordinates may not match some local control performed by conventional traverse in even fairly recent years. Two vertical datums exist in North America and they cannot be mixed. Final

results must produce coordinates, which are precise relative to one another and from which a route project can realistically be developed.

3.7.9 GPS Control Survey Final Products

A completed GPS control survey should include the following:

a) Mission planning results for all sessions showing satellite visibility (including any obstructions present), PDOP plot and a plot of the number of healthy satellites available.

b) A copy of all fieldbook information, which should include descriptions, sketches and rubbings of all points for each session. The height of instrument (measured twice by independent methods), temperature and pressure should be included in the fieldbook information for each occupation.

c) All raw receiver file information should be provided in RINEX format or such other format approved by MHD.

d) Any edited raw receiver files should have changes noted.

e) All output files from vector processing should be included. Any solutions used which were not fixed solution, or a ratio of 100, should be highlighted and detailed as to why they were used.

f) A listing of the minimally constrained least squares network analysis where the "best" control point's coordinates were held as fixed. Loop closures should be provided if deemed appropriate.

g) A listing of the constrained adjustment highlighting the control that was used. If some occupied control coordinates were not used, the cause for rejection should be explained.

h) Any elevations produced should be clearly identified as GPS-derived and not given any class or order. The procedure for obtaining elevation (GEOID 93, GEOID 96, published Geoid/ ellipsoid separation, etc.) should be described in detail.

3.8 PHOTOGRAMMETRY

Photogrammetry is the use of aerial photographs for measurement and mapping purposes. Traditional applications for most route projects involve:

a) Large-format calibrated aerial cameras with a specific mount in an airplane;

b) Processing of photographs in one-to-one production of images on film or glass diapositives specifically designed for minimization of distortion due to temperature, pressure, and humidity changes during production;

c) Adequate ground control, which is either targeted, or photo identifiable;

d) A measurement and ensuing least-squares analysis process called aerotriangulation, which validates the ground control and densifies it as needed for use in stereoplotter orientation and map compilation;

e) Stereoplotter orientation based on the densified ground control which resolves the relation of the photos to each other and the ground at the time of exposures and provides a check on the quality of the aerotriangulation;

f) Compilation of the desired features, which could include planimetric features (line and point symbology), contours, cross sections, profiles, break lines, spot elevations, and text information;

g) Clean-up of compiled information to make it topologically pleasing, e.g., an edge of driveway should not extend past the outline of a house;

h) Addition of field survey data that could not be collected photogrammetrically due to obstructions, cover, shadow, etc.;

i) Production of final products, which could include translation to other digital map formats and/or hard copy output.

3.8.1 Camera

The typical camera used for photogrammetry is metric with a format of about 227 mm x 227 mm and a focal length of 152.4 mm (nominally 6 inches). The exposure system has a film flattening mechanism so that during exposure the film is flat and perpendicular to the line from the camera principal point to the rear nodal point of the lens. A large magazine for film storage is required to accommodate the large number of exposures, which can occur during a mission. The camera should contain at least eight fiducial marks in the corners and sides of the picture area for measurement of film shrinkage or expansion. A mechanism to correct for airplane movement during exposure is highly desired.

The camera should be calibrated by the U.S. Geological Survey approximately every two years. Calibration reports provide calibrated focal length, fiducial mark coordinates, principal point location, resolution, and radial lens distortion.

3.8.2 Photogrammetric Flight Mission

Photography is taken at a flying height, which is a function of the accuracy of the desired product and should be within 5% of the desired scale.

Since adverse weather will most likely affect picture quality, marginal days for aerial photography should be avoided. Low sun angle, a function of time of day and season, should also be avoided so that the image quality should not be adversely affected by shadows or reflection. Obstruction by leaf cover, a function of season, should also be avoided.

A suitably equipped airplane flying as straight as possible along a flight line captures aerial photography. Since stereo coverage (each point on at least two photos) is required, overlap (endlap) between successive photos along the flight line should generally be between 60-65% with deviation of more than 5% considered unacceptable. To ensure proper stereo viewing, the camera axis should be as near vertical as possible, with tilts along the flight line or perpendicular from it of more than three degrees usually considered unacceptable. Rotation of the flight line between successive photos, or from the overall intended flight line direction, of more than five degrees is considered unacceptable.

Substitute photography, obtained in place of unacceptable photography, shall comply with the original requirements. Overlap to the original accepted photographs must meet the minimum specified limits but can overlap more extensively at the ends to ensure no gaps in the data. Numbering of substitute photography shall continue

in an unbroken sequence from the last number used in the original photography.

Some projects require multiple flight lines to satisfy job requirements. The flight lines are usually parallel and overlap between ground coverage of adjacent flight lines is generally 20 to 35 percent. Multiple route geometries for the same project (intersecting roads) may require multiple flight lines, which are non-parallel. To ensure no gaps in coverage, standard practice is to have 100% overlap of these lines in the intersection area.

3.8.3 Film Production

Film production should result in so-called diapositives on either glass or film. All prints should be contact prints, produced at the same size as the negative. The diapositive image should be distinct and clear so that all desired planimetric and topographic features are easily measured. The processing should minimize any shrinkage or expansion of the film.

To ensure easy management of the information, a photography index should be prepared. Consecutive overlapping photographic prints are edge-matched near the center of their overlap area and attached together using staples, glue, etc. The photos may be trimmed in the overlap area to ensure neat edges. The photos should be labeled with project number, date of flight mission, job-specific photo identification and any other pertinent information deemed appropriate. The labeling must be in a consistent location on all photos.

3.8.4 Ground control

Horizontal and vertical ground control is required in the coverage areas for any photogrammetry project. MHD experience with photogrammetry operations indicates a need for horizontal control every 5-6 photos along a flight line, with vertical control every 3-4 photos. The need for vertical control is due to the weaker geometry of aerotriangulation and analytical stereoplotter orientation for mathematical solutions in the Z dimension. If aerotriangulation is not being used, a minimum of three horizontal and four vertical control points are required in every stereomodel to provide the required geometry and a redundant check. The ends of flight lines should contain both horizontal and vertical control.

Since route projects are often single flight lines, all of the exposure stations are in a near straight line. This geometry requires that some of the vertical control be located at the outer

edge of the photography, perpendicular to the direction of the flight line. These points are often called wing points and require ground survey to be performed off the project right-of-way as the route is usually parallel to the flight line and near the middle of the photography.

Ground survey of the control points should tie in all control stations in one contiguous survey network, or ensure only high-order control stations are used. Utilizing independent surveys based on different control does not confirm the reliability of control coordinates to each other, so it is important to create a survey "network." Except in unusual circumstances, all surveys should tie to current standard horizontal and vertical datums and all coordinates produced should be based on those datums. Datums are discussed in Section 3.4, Conventional Field Data Processing.

Care must be taken to ensure that control will be visible on the photographs at the time of the flight mission. Shadows from trees or other structures can easily make control unidentifiable, as can vehicles moving over control points located on a paved surface.

Targeting of control points is highly recommended to ensure easier identification on the photographs. Target size is a function of the flying height of the photography. On suitable surfaces (pavement, sidewalk, etc.) targets are painted in a color that contrasts with the surrounding area; targets of durable material (wood, fiberglass, strong cloth, etc.) are used on grass or dirt to ensure that grass will not grow to obscure the target or rain will not cause runoff to cover the target in the interval between targeting and flying. Targets are usually in the shape of a "+" or a "T" with the intersection precisely positioned over the control point. In the case of a vertical control point where the target is elevated over the survey point, one must ensure that the mapping is based on the target elevation corrected for its offset to the survey point.

Control points should be explicitly described in a fieldbook so the position is uniquely defined. The location of control points should be pin pricked on prints and highlighted with grease pen. A circle with a marker should be made on the back side of the print around the pin prick location, along with a point identification name or number, and short description if needed.

The control survey should follow standard survey procedures as described in Section 3.2, Field Operations With Conventional Survey Equipment. GPS is especially suited for such use since both it and control targeting require open areas. If GPS is used for vertical control, one must ensure that the derived elevations from ellipsoidal height differences are of suitable quality for the required accuracy for the project. If trigonometric leveling is used for elevation determination, care also must be taken to ensure it is

of suitable quality for the project. Differential leveling for vertical control will be required for many MHD projects to ensure required accuracy.

Airborne kinematic GPS can be used in the measurement of exposure station positions as control to reduce the amount of ground control needed. However, there must be sufficient ground control so that aerotriangulation can be processed and used as a check on the kinematic GPS positions and to ensure required accuracies are maintained.

3.8.5 Aerotriangulation

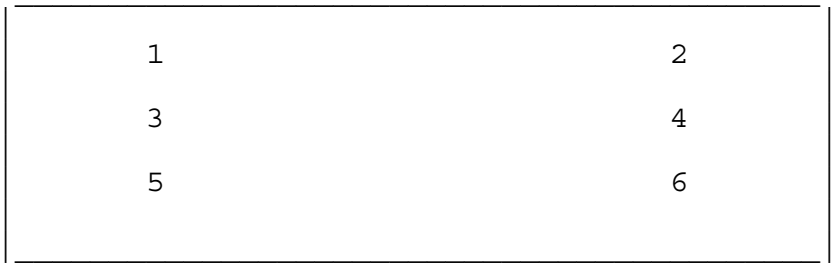
Aerotriangulation is a procedure, which validates the quality of the ground control, coordinates and densifies that control to where it is suitable for the collection of the positional and attribute information desired for the project. The quality of the aerotriangulation results is not only dependent on the quality of the ground control, but also on the quality of the photogrammetric measurements, which are part of the aerotriangulation process. Photogrammetric measurements in this case are precise measurements of the x,y photocoordinates of points based upon a coordinate system aligned to the fiducial marks, which places its origin near the center of the photograph and whose x axis is between the side fiducials which are along the flight line axis.

Measurement of photocoordinates is usually performed on an analytical stereoplotter but could be performed on an instrument called a comparator whose sole purpose is coordinate measurement and not map compilation. Resolution of measurements is generally in the 0.001 mm range with accuracy tending to be in the 0.003-0.010 mm range. Instruments of lesser measuring quality should only be used if results are obtainable that meet the accuracy requirements of the project. Coordinates are usually measured in stereo as it is a quicker measuring process but can be measured monoscopically if all points are monoscopically photoidentifiable.

To obtain photocoordinates, fiducial marks are first measured. This allows the arbitrary coordinate measuring system of the stereoplotter to be transformed to a photocoordinate system. Since the calibrated fiducial coordinates can be compared to their measured values during that process, a correction for shrinkage and expansion of the diapositive can be applied. Control extension is performed through identification and measurement of "pass points". These must be user-picked photoidentifiable points, but in many cases are marked positions using a process called "pugging". A pug is an artificial mark in the emulsion of the photograph usually made by a very small drill bit on a pug machine. Pug points are usually picked by stereo viewing on the pug machine, at locations which

ensure the necessary geometry for a mathematical solution in aerotriangulation and are also easily viewed in stereo for precise measuring purposes. If the points will be measured in stereo the pug is only made on one photo as the mark will "appear" in an adjacent photo through stereo viewing.

Aerotriangulation requires at least six points to be measured in every stereomodel, positioned approximately as shown below.



Points 1, 3, and 5 should be measurable in the adjacent left stereomodel, and likewise points 2, 4, and 6 should be measurable in the adjacent right stereomodel. The ability to perform this operation is ensured by the 60-65% endlap rule. If the job has multiple flight lines that are adjacent, exterior points (1, 2, 5, and 6) should be measurable. A control point near one of these locations can be substituted as a pass point in lieu of identifying an additional point.

Points 1, 3, and 5 are near the center of the left photo in the X direction and would be pugged (if this process is used) on that photo. They would then be able to be viewed in stereo in the two models that it is part of. Similarly points 2, 4, and 6 would be pugged on the right photo.

The first check on the quality of the measurements is that, except for end photos on a flight line, a point is measured in at least two stereomodels, i.e. measured twice so coordinates can be compared and generally averaged if deemed acceptable.

The mathematical component of the aerotriangulation is a three step process though some processing systems take only two steps.

A) Stereomodel Joining

During the collection process, each stereomodel produces 3-D coordinates of measured points called model coordinates, which are derived from photocoordinates. The model XYZ system is simply an assumed system necessary for processing, but not used in the final product.

Independent stereomodels along a flight line are "joined" together based on common points and 3-D coordinate transformations to make one uniform coordinate system. The fit of the coordinate transformation residuals is an indicator of the quality of the photogrammetry measurements.

B) Strip and Block Adjustment

The flight line's contiguous model coordinates for control points are compared to their ground control coordinates permitting a 3-D coordinate transformation of all model coordinates into the ground coordinate system. To account for systematic errors in the control and the systematic error buildup of independent model coordinate joining, polynomial corrections (usually no higher than second degree) are applied as part of the 3-D coordinate transformation. Since the control should be redundant, residuals in ground coordinate units indicate the "fit" of the process. As part of this process, usually called strip adjustment, errors in control coordinates can usually be identified.

If multiple flight lines exist, the tie points between flight lines are used in 3-D coordinate transformations with polynomial corrections to indicate the fit of the separate strip adjustments and attempt to produce a more uniform result for the ground coordinates of all measured points.

C) Bundle Adjustment

This procedure is a simultaneous least-squares analysis of all photogrammetric measurements and control coordinates, and theoretically, is the most effective means for analyzing final results and producing the best possible final coordinates. The processes detailed in (A) and (B) above serve as an information check and set up some necessary intermediate information required for this step. In addition to solving for all final ground coordinates, this procedure solves for all camera exposure positions and the angular orientation of the camera axis at the time of each exposure relative to the ground coordinate system. These results validate that appropriate flying heights and camera orientations existed during the flight mission.

Because of the high precision of the photogrammetric measurements (0.003-0.010 mm), holding ground coordinates (which contain random error) fixed can affect the output accuracy indicators of the photocoordinates. Suitable error estimates should be placed on the photocoordinates and on the ground control coordinates to produce suitable least-squares analysis statistical indicators from the bundle adjustment. Final results should place the overall photocoordinate root-mean-square (rms) error in the 0.003-0.010 mm range, while the ground coordinate rms errors

should reflect the expected reliability of their values based on the survey procedures used in their derivation.

Based on suitable aerotriangulation results, all points measured during the aerotriangulation process will have ground X,Y,Z coordinates resulting from a simultaneous analysis of all ground control and photogrammetric measurement data. Due to the location of pass points in stereomodels, at least six points have ground coordinates, a redundant solution for the stereoplotter orientation required for mapping. Since the control also exists in multiple stereomodels and possibly flight lines, there is assurance that in map compilation the X,Y,Zs of collected data in one stereomodel will closely match (called edge match) their corresponding positions in adjacent stereomodels.

3.8.6 Types of Stereoplotters

An analog stereoplotter without encoding has no computer communication. All orientations are performed manually and all information must be compiled in hard copy format. Since this type of compiling dramatically limits the accuracy of the produced information, and it is inaccessible to computer systems, its use is not recommended.

An analog stereoplotter with encoding enables limited communication between the computer and the stereoplotter. Computer software can enhance the orientation process, although it is still performed through a series of operator-controlled dials. Data can be entered directly into the computer with the limitation that the computer generally cannot drive the plates which the photographs lie on. Aerotriangulation is usually not performed on these instruments. While this process could be used if it is verified that the encoding is precise enough to meet job requirements, its use is not recommended.

An analytical stereoplotter is one in which all orientations are computer driven, the computer can drive the stereoplotter plates, and the data collection is completely digital though all measurements are made by the operator. It must be verified that the measuring resolution of the stereoplotter is precise enough to meet the job requirements.

With a digital stereoplotter ("soft copy") photographic diapositives are replaced by computer images. While the image could be produced by some non-film based raster collection system, most digital images are produced using a scanning process of the image from the diapositives. One needs to ensure the scanning resolution is of a size that permits the required accuracy requirements to be achieved, or that any systematic errors in the scanning process do not cause

similar problems. Digital systems can usually replace some of the human operations through image recognition, infinite zooming capability, and by allowing the collected data to be superimposed on the raster image. Due to the large file size of scanned images at small resolution, and the need for efficient management of the data files, large computer drives and rapid computer processing speeds are required.

3.8.7 Stereoplotter Orientation

Stereoplotter orientation involves recreating the camera exposure geometry (position and altitude) and the creation of a ground coordinate system derived from the stereomodel.

Orientation is a three step process, though the last two are often integrated into one measuring process.

a) Inner Orientation - The process of placing the photographs on the plates in the stereoplotter and measuring the fiducial mark locations. A 2-D transformation (unique to the left and right photos) produces photocoordinates and residuals since the transformation is a redundant solution.

b) Relative Orientation - The same point in the left and right photo is measured for six distinct points usually in the general locations of the six standard pass points. More than six points can be used for a more redundant, and hopefully better solution. This solution resolves how the exposures were angularly related to each other at the time of exposure and on computer based systems results in the production of model coordinates. Residuals again indicate the quality of the measurements of this procedure.

c) Absolute Orientation - All control points in the stereomodel are measured and a 3-D conformal transformation converts model coordinates into ground coordinates, with residuals if the solution is redundant in nature. On analog systems this is commonly referred to as a scaling, rotation, and leveling procedure. The residuals (in ground units) should be of sufficient quality to meet the job accuracy requirements.

   A measuring mark is moved through the stereomodel. After absolute orientation, the X,Y,Z position of the mark changes as it moves. To measure a position, a mark is placed on a point's image. The position is in 3-D since it is viewed in stereo.

The merging of relative and absolute orientation, usually called outer orientation, is performed by measuring control points as part of the relative orientation process. If aerotriangulation has been used, analytical and digital stereoplotters can perform automatic

relative and absolute orientation from the aerotriangulation results and only redoing inner orientation will be required prior to mapping. On some digital stereoplotters, inner orientation does not have to be reperformed when aerotriangulation is complete since there is no placing of diapositives back into an instrument.

Inner and relative orientation residuals are in photocoordinate units and thus should rarely exceed 0.010 mm. Absolute orientation is in ground units and a general rule is that these residuals should not exceed 50% of the expected accuracy of the final product. For example, if a DTM is expected to be within 0.20 m relative to a field-checked position, the vertical residuals in absolute orientation should not exceed 0.10 m.

3.8.8 Topographic Data Collection via Photogrammetry

Once stereoplotter orientation has been completed, any form of topographic information can be entered. Such topographic map information generally refers to items such as planimetric features (line and point symbology), contours, cross sections, profiles, break lines, spot elevations, and text information. If a job requires a digital product (usually in addition to hard copy output), MHD file format should be followed.

A) Planimetric Features

Job requirements will determine what planimetric features need to be collected and MHD symbology standards should be adhered to. Some job specifications may define what symbology will be used to define points and lines, while for other projects it may be left up to the photogrammetrist to come up with a standard symbology and convey the linkage of the symbology of the data to what it actually represents.

Symbology for points (manholes, power poles, trees, etc.) are at a given size based on the desired scale of the final product. Both color and line type make up a symbol. Likewise, textual information is generally of a size which is realistic for the product's map scale.

Lines can be either straight, curved, or a combination of the two. In most cases "curved" implies a spline representation of the point information. The curve can imply circular, such as the case where only three points make up a curved line, or when two straight tangents are each defined by two points, and intermediate to them is a curve point. This case produces a circular arc between the two tangents.

Map information should be seamless across map sheets and across

stereomodels. When compiling a stereomodel, which overlaps one previously compiled, it is important to verify that no significant misclosure in planimetric position or in elevation exists for the same line or feature. The same is true for a spot elevation. If a misfit between stereomodels is well within random error limits, common practice is to "fit" the latter to the data already collected. A misfit exceeding random error limits is generally caught in the aerotriangulation and/or orientation process. If a large misfit is found during map collection, mathematical solutions in that region should immediately be checked.

B) Contours

Collection of contours by photogrammetric means has traditionally been an actual 3-D (stereo) view tracing of the contour line. The contour line is made topologically correct when it meets items such as roads, buildings, etc.

Prior to the digital map era, the stereoplotter operator ended contours at planimetric features like buildings. Digital mapping allows the creation of many different types of maps, one of which is only contours. Such contours would be continuous, even passing through a building location. Software allows continuous contours at features like buildings to be "clipped" if that type of map is desired.

Many contours are now derived from DTMs, which are a combination of spot elevations and break lines. DTMs afford a better 3-D analysis of data and output of various elevation information. An internal check of the quality of contours derived from a DTM is made by comparing those to contours traced conventionally. An external check would involve comparing field cross sections to those derived from the DTM, or comparing contours derived from a field survey with those developed photogrammetrically. Accuracy requirements would be a function of the intended purpose of the data.

C) Cross Sections

In open areas, the collection of cross section information by photogrammetry can dramatically speed up tedious field data collection while at the same time eliminating danger due to traffic in the field collection process.

Photogrammetric cross section software requires the identification of stationing along a defined baseline. The desired interval for cross sections and the distance to be cross sectioned left and right of centerline is also input by the user. Software will utilize this information on analytical and digital

stereoplotters to move the operator to the proper location to be measured.

The operator can set up the software to drive the measuring mark along the cross section line as the mark is manually set to the correct elevation, or the software can limit the operator to moving the mark only perpendicular to the centerline. When obstructions or shadows prevent measurement directly along the cross section, software permits the operator to move left or right of it to collect an elevation, which will be "moved" back to the cross section line.

Software generally allows the operator to define more cross sections at important intermediate positions such as intersections, high points, low points, etc. The cross sections should be able to be output in hard copy at a desired scale, or translated to a desired format for engineering design purposes.

D) Profiles

A cross section is a profile at a defined station perpendicular to the centerline. A profile is simply a generic form of elevation representation along a defined line. Profiles are compiled photogrammetrically in the same way as cross sections.

Typical profiles required on route projects may include centerline, edge of pavement, top or bottom of curb, bottom of ditch, etc. It is usually important to identify important planimetric information along a profile such as edge of driveway, manhole, etc.

E) Break Lines and Spot Elevations

While a more thorough discussion of this topic is found in Section 3.2, Field Operations With Conventional Survey Equipment, break lines and spot elevations are the type of data included in creation of a DTM. A DTM which is developed strictly from spot elevations infers that no sharp breaks in grade occur anywhere in the ground surface. Since this is rarely the case on route projects, sharp breaks which should be collected as break lines occur at bottoms and tops of ditches, at other abrupt changes in the ground surface, and at man-made breaks in grade such as top of curb, etc. The DTM surface will noticeably "break" at these lines, while in areas where only spot elevations exist the surface will look smoother.

Photogrammetric collection of spot elevations (X, Y, Z mass points) is usually performed on analytical or digital systems by having software dictate a user-defined interval throughout the stereomodel in grid form. The density of the grid is a function

of the required accuracy and the undulation level of the terrain. The user can ignore a point or move a point slightly for measurement purposes if a grid intersection is obscured due to trees, shadows, or other reasons. The selected points are usually displayed graphically. Break lines are simply a special line type and are collected as any other line type, though special care must be taken to measure precise elevations during the process.

Most DTMs are generated from creation of a triangulated irregular network (TIN) of the spot elevations and the break lines. Contours, perspective views, profiles, and cross sections can then be derived from the DTM. As an internal check to verify the internal and external quality of the DTM, it is common to collect some profiles and/or cross sections from the same information that the DTM was created from and compare them to those derived from the DTM. To check externally, collect profiles and/or cross sections by conventional field procedures and compare them to those derived from the DTM. The accuracy requirements will be a function of the intended use of the data.

F) Text Information

As with any ground-based topographic survey, descriptive information is often required in addition to map symbology and positional information. The textual information should have consistent placement relative to the positional information, with a size compatible with the desired hard copy scale. MHD specifications should be strictly adhered to.

Information required for cross sections, stationing, and profiles tends to be fairly standard. The difficulty lies in how much textual information is required to augment planimetric data which cannot be conveyed through symbology. Consistency for a given project is required, and the type of job will dictate the amount of textual information required.

3.8.9 Map Clean-up

The stereoplotter operator is usually not responsible for final-product topographic information. It is not considered time and cost effective for the operator to spend much time making map data topologically pleasing. For example, for compilation of a driveway added to an already compiled building, the end of the driveway will most likely not end precisely at a building edge line. Similarly, the end line for a building outline may fail to close exactly with the initial point. A more difficult problem is to ensure that a contour line crosses the edge of pavement at that line's elevation.

Manual map cleanup is usually performed at a digitizing tablet, where problems are visually searched for and corrected. It can be a time consuming process and conditions that need correcting can be easily missed.

Software can be used to perform map cleanup automatically, and can work well if effective feature coding is utilized. For example, all driveway feature codes whose end-of-lines come within X m of a line designated by a house feature code should intersect perfectly. All house feature code end-of-lines which do not intersect or connect with another line should be intersected with another end if within Y m. The values for X and Y would be determined by the accuracy at which the data is being collected. While such software will not find every possible type of clean-up required, it highly automates the process for the most common types of clean-up.

3.8.10 Integration with Field Survey Information

It is rare that all required information can be collected photogrammetrically. As examples, an edge of pavement may be obscured by tree cover, underground utilities will not be seen from an aerial photo, an object may be difficult to measure because of high reflectance and the distance from bottom to top of curb is often too small to see photogrammetrically. This type of data must be collected by conventional ground survey and added to the photogrammetry data. In such cases, the same control must be utilized to integrate the data correctly.

Field survey should also be used to check both the positional and attribute photogrammetric data. Positional checks can include horizontal position of planimetric features, spot elevations of distinct points, profiles, and cross sections. It is most important that the same control used for the photogrammetry is used for the field survey and that field-checked spot elevations, profiles and cross sections are correctly located. An improperly located field check cannot match the photogrammetric data since each set of data will represent a different ground position.

Confirming attribute information involves field checking that the plotted information for a point or line is correct. For example, a stone wall may have been inadvertently coded as a wooden fence. Also, the surveyor needs to check for missing point or line information.

3.8.11 Final Products

Hard copy output of collected data is an important job product, since the end user may not have the necessary hardware or software to work the data digitally.

The scale, type of map media (paper or mylar) and the type of output are important needs, which must be defined before the project begins. The end user should also define layout size, title positioning, scale representation, north arrow, etc. MHD has specifications for these parameters.

When digital (computer based) products are desired, the photogrammetry system's digital output may not be in the same format as the software utilized by the end user for purposes such as planning, engineering design, traffic, right-of-way, etc., and translation between formats is necessary. Accurate and efficient translation that provides a correct end product must be ensured.