Energy, Mines and Resources Canada

Énergie, Mines et Ressources Canada

Supplementary Control Surveys

Canada Centre for Surveying
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SUPPLEMENTARY CONTROL SURVEYS

GEODETIC SURVEY OF CANADA
CANADA CENTRE FOR SURVEYING
ENERGY, MINES AND RESOURCES CANADA

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FOREWORD

Supplementary control surveys are those which augment the primary horizontal and vertical control networks. Their principal purposes are to increase network density and to provide survey control necessary for mapping. These purposes often overlap, giving rise to the term "multi-purpose" control. Supplementary control surveys are classified in accordance with SPECIFICATIONS AND RECOMMENDATIONS FOR CONTROL SURVEYS AND SURVEY MARKERS (Specification Series 1978).

This manual describes the survey techniques and procedures used by the Geodetic Survey Division, Canada Centre for Surveying, Department of Energy, Mines and Resources, Canada, in establishing supplementary control surveys. It includes the planning and preparation of field survey projects, field methods and techniques used and the processing of survey data.

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1 *SURVEY CONTROL FOR MAPPING*

1.1 Control Requirements

1.1.1 Introduction

Survey control needed for the National Topographic Series (NTS) mapping program is supplied by the Geodetic Survey Division. Control requirements are determined by Topographical Survey Division, the National mapping agency.

The main thrust in National mapping is completion of the NTS 1:50 000 scale coverage and recompilation or revision of existing sub-standard or out-of-date maps. Reference to mapping control in this chapter implies a scale of 1:50 000, unless otherwise stated.

The density and placement of control needed for a mapping project is subject to a number of variables; for example, the size and shape of the block to be mapped and the scale and direction of the mapping photography. These factors determine the approximate locations of the required control.

The surveyor plans his survey to meet these requirements. Minor changes may be made in the field as to the placement and requested density; major variations require prior approval by the Topographical Survey Division.

1.1.2 Horizontal Control for Mapping

The minimum horizontal control required for a mapping project is one point in each corner of the block to be mapped. The points should be in the first and last models of the photographic flight lines. This also applies to irregular-shaped areas. Additional control points are placed outside the perimeter of the area to be mapped, as close to the edge as practicable, and are spaced at a distance not exceeding eight times the distance between exposures of the compilation photos. There are usually additional existing points that may be used to verify the aero-triangulation block adjustment.

All horizontal control established for mapping must include redundant field observations and be integrated with the geodetic network. The accuracy specifications are:

DISTANCE BETWEEN CONTROL POINTS	ALLOWABLE HORIZONTAL ACCURACY (1d)
1 km to 8 km	1.0 m
8 km to 30 km	1.4 m
30 km or greater	1.6 m

Table 1-1 Horizontal Control Accuracy for 1:50 000 NTS Mapping (GSD 80-5)

1.1.3 Vertical Control for Mapping

Vertical control points for mapping are normally established in straight lines running at right angles to the photographic flight lines and spaced four to six overlaps apart. Two vertical control points are required for each stereo photo model to be controlled. These are normally placed in the common overlaps between flight lines. If it becomes necessary to position a control point outside the common overlap between adjacent flight lines, an additional point is required on the second flight line. Control point images should not be less than 2.0 cm from the edge of a photomodel or 2.5 cm from the corner. In addition, vertical control is needed on the first and last model of each flight line. Other factors, such as the size and number of lakes, are considered when deciding how much control is required. Large lakes can be used in levelling a photogrammetric block adjustment even without knowing their elevations.

When selecting a mapping control point the surveyor must keep in mind that water surfaces themselves cannot be used by the photogrammetric machine operators for vertical control. They must be able to discern some relief on the model. In addition the water level at the time of the survey may differ from that when the mapping photography was taken. This can sometimes be ascertained by comparing the actual shoreline with the one on the mapping photo. A note regarding this should be made on the photograph and in the notebook.

For the Northern Aerial Survey Data Base (NASDB) the vertical control accuracy, with respect to the control being used, varies according to the contour interval and the extent of tree cover.

CONTOUR INTERVAL	TYPE OF COVER	VERTICAL CONTROL ACCURACY (1σ)
10 m	clear	1.5 m
10 m	wooded	3.0 m
20 m	clear	3.0 m
20 m	wooded	4.0 m
40 m	--	4.0 m

Table 1-2 Vertical Control Accuracy for NASDB (GSD 82-7)

For the Southern Aerial Survey Data Base, the required standard deviation for vertical control is 0.6 m (GSD 82-7).

1.2 **Site Selection and Preparation**

1.2.1 Introduction

Field conditions often dictate that survey station sites be prepared prior to the actual surveying. On projects for which helicopters are being used for transport, for example inertial surveys, for which the smooth and uninterrupted progress of the survey is critical, it is essential that site preparation be completed before the survey begins.

1.2.2 Site Selection

Satisfactory site selection implies the choice of a location that is suitable for the subsequent surveying and mapping process. It also implies that the site is suitable for helicopter landing and take-off, if required, and that the station can be adequately marked, targetted and photo-identified. On some surveys it is also essential that sites be intervisible to permit angle and distance measurements.

1.2.3 Access to Private Property

Many Geodetic horizontal control stations are on private property. Station sites may not be visited or work done without the permission of the landowner or his representative. In the course of the work, surveyors who deal with landowners or their representatives must act fairly and diplomatically.

They must identify themselves, state why access is required, how long work will continue at the station and negotiate payment, if any, for right of access, rental of land and for trees that may have to be cut, etc. If dealt with diplomatically, most landowners will permit access without payment. It has been recent practice at Geodetic Survey to pay landowners for access when such access is detrimental to the use of the land. Even when large survey towers are erected for a considerable length of time, payments exceeding $100.00 are rare. All payments must be in the form of rental of land or purchase of materials - never as compensation for damage. Field officers are not authorized to pay compensation of any kind.

1.2.4 Site Preparation

When helicopters are to be used on the survey, a landing area at least 25 m in diameter should be completely cleared of bush and obstructions to ground level. A level and firm landing area is essential. The use of landing platforms should be avoided but, if necessary, should measure at least 3.5 m by 3.5 m, and must be secured to withstand the downdraft of a helicopter. In addition to the clearing of a landing area, a take-off and landing path not less than 25 m wide should be cleared of obstructions above a 15° angle from the centre of the clearing. The take-off path should be oriented into and away from the prevailing wind, usually NW and SE. If the station is to be surveyed by the Inertial Surveying System (ISS), the survey markers should be placed on level ground so that the helicopter can land in any direction, facing the survey marker, no further than one metre from the marker.

1.2.5 Survey Markers

Survey markers used on supplementary control surveys include those recommended in "Specifications and Recommendations for Control Surveys and Survey Markers", (S & M, 1978) and also a number of other markers which, though adequate for mapping control, may not meet the rigid requirements for higher-order surveys. This permits more flexibility in the choice of a survey site by providing a broader range of marker types suitable for different ground conditions.

A more comprehensive listing of marker types is in Appendix B (page B-1).

If a station is required at a site where an acceptable marker already exists, the existing marker should be used to avoid confusion. A unique number will be assigned to the existing marker (unless it already has one) and other station numbers on it will be used as the station name. Reference markers are not generally installed unless so specified in the field survey instructions. Azimuth stations will be marked with appropriate horizontal control survey markers.

Markers for ISS stations are not to be placed within 10 m of any obstacle that protrudes more than 1.5 m above ground level, e.g. a fence, etc., nor shall any marker be placed beneath overhead wires if there is a possibility that the stations will be visited by helicopter or if a tower may be required.

Tops of survey markers placed in areas that may be cultivated should be at least 30 cm below the surface and should be referenced.

1.2.6 Marker Identification

Accurate photo-identification of survey control points intended for mapping is essential. The points must be identifiable on the aerial photography to be used in the compilation of the map. This is accomplished by targetting the points, photographing them from the air and transferring the target images stereoscopically to the high altitude mapping photographs (diapositives). This is discussed further in 1.2.7 and 1.3.

1.2.7 Targetting

Identification photography of targets must not be covered by shadows. If the control point is in a clearing surrounded by trees, the clearing must be sufficiently large so that shadows will not fall on the target. A target is useful only if its image can be transferred to the mapping photography. This may not be possible if the terrain has undergone major changes as the result of forest fires, seasonal snow cover, logging, mining, erosion, etc., which have occurred since the mapping photography was flown. The mapping photography should be checked stereoscopically, on site, to verify that the site can be identified and plotted. It may be necessary to survey an auxiliary point for targetting if the area of the survey station is not suitable. If this is necessary, a target will be placed on the main station and one of different shape at the auxiliary station. A sketch showing both targets is to be drawn and must form a part of the permanent record of the station.

Some terrain types are unsuitable because targets placed on them may not be visible on the identification photography. Generally, land types which appear white on photographs, such as sand, some types of rock, caribou moss, are unsuitable unless a dark target is used.

Experience has shown that white is the preferred colour for the target material, and best results are obtained when it is used on a dark background. If a black target is used on a white background, its size should be doubled. There is image spread in the negative emulsion of the film and a black target's image on a white background is smaller than that of a white target of equal size on a black background.

Three target shapes recommended are: panels in the shape of the letter T, a cross, and the letter Y (legs at 120°). The Y shape is preferred as it can be most easily seen on a photograph.

Target size can be calculated from the formula:

$$tg = \frac{ft}{f} \qquad (1\text{-}1)$$

where tg is the target diameter in metres;

h is the flying height of the identification photography in metres;

t is the diameter of the image size that can be seen on the photograph in millimetres (usually 0.5 mm, but as small as 0.3 mm under ideal conditions);

f is the focal length in millimetres.

For additional information on target dimensions and photography specifications refer to Appendices A1 to A4.

Two types of target material generally used are 8-gauge vinyl sheets and cotton cloth. Successful experiments have been carried out using biodegradable paper where the targets must later be removed. This material can be obtained on special order through Technical Field Support Services (TFSS). Vinyl is much heavier and more difficult to handle than the cotton cloth. The latter is preferable when helicopter transport is required. The reflective qualities of the materials are about the same.

The target panels should be carefully aligned with respect to the survey marker. A tripod spread out on the ground with legs extended helps in placing Y-type targets at the proper angle and distance from the marker.

1.3 Identification Photography

1.3.1 Introduction

Stereoscopic photo coverage of mapping control points is recommended to facilitate accurate transfer of the target images to high-altitude mapping photography. This coverage usually consists of short flight lines over the target so that stereo-pairs of photos at a predetermined height are obtained.

1.3.2 Cameras

Several types of cameras are used for identification photography; a brief description of each follows.

The Hasselblad MK70 is equipped with a Biogon 60 mm lens with auto-diaphragm control, MK70/200 magazine, auto film winder, intervalometer, and reseau grid. The reseau grid is a glass plate on which is etched an accurate grid of small crosses that are reproduced on the negative. These etched lines are calibrated and provide a means to measure film distortion as well as a means of relating the position of the target to identifiable points on the photograph photogrammetrically. The identification photography taken with the Hasselblad, with reseau grid, can be used directly in the aerotriangulation adjustment. This is useful in cases where the target cannot be transferred but where the photo has other features which can be identified on the mapping photography. The film wind motor is powered by two internal rechargeable 6V batteries. The shutter speeds can be varied from one second to 1/500 of a second and the focussing range is from 0.9 m to infinity. The magazine holds enough Plus X film for 150 exposures, and has a pressure plate. The shutter can be released either by a release button or by the intervalometer at predetermined intervals.

The Vinten camera has a 44 mm focal length, f2 lens, and a fixed shutter speed of 1/1000 of a second. The magazine holds 59 m of usable thin base film, sufficient for 890 exposures. An intervalometer is supplied with the camera and must be connected to the camera to take pictures at timed intervals. Single frames can be taken using a release button on a cable. The film advance is driven by a D.C. motor powered by a 24-volt battery. The lens is focused at infinity. A General Electric exposure meter Type PR3 is suitable for use with the Vinten camera.

The Pentax MX has a focal length of 20 mm and uses standard 35 mm film of 36 exposures. The camera is equipped with an auto film winder that can be attached to the camera and is powered by an internal battery. Shutter speeds vary from 1 second to 1/1000 of a second.

The Ricoh Hi-color 35 has a focal length of 35 mm and uses standard 35 mm film of 36 exposures. The shutter speed can be varied from 1/30 to 1/300 of a second and the diaphragm is controlled automatically.

The Vinten and Hasselblad cameras use Tri-X and Plus-X aerographic film, 70 mm perforations, type II. The film must be hand-wound on spools for the Hasselblad camera. Kodak Tri-X film is recommended for the 35 mm cameras. Film should be stored in a dry, cool place and should not be used beyond the date of expiry. The film can be frozen to stop deterioration, and the time that it is in cold storage can be added to the date of expiry.

A mount has been constructed for the Hasselblad camera that fits under the Jet Ranger and Hughes 500 helicopters. The Vinten and 35 mm cameras are usually mounted internally in the helicopters and the pictures taken through an aperture in the bubble. Photo-identifications from fixed wing aircraft are normally carried out using aircraft with camera hatches.

1.3.3 Aerial Photography

The scale of the identification photography should not exceed 2 times the scale of the mapping photography. However, a maximum of up to 2 1/2 times can be tolerated. The flying height (above ground), required to obtain the desired scale, can be computed from the formula:

$$h = \frac{f}{s} \qquad (1\text{-}2)$$

where h = the flying height of the aircraft in metres above the ground

s = the required scale ratio of the photography

f = the focal length in metres

For example:

$$f = \quad 020 \text{ metre (Pentax)}$$

$$s = \frac{1}{30000}$$

$$h = \frac{.020}{1/30000} = .020 \times 30,000 = 600 \text{ meters}$$

For a graph of flying height in feet versus photo scale, see Appendix A (page A2-1).

Identification photography should be taken at normal cruising speeds, with exposures timed to produce 80% forward overlap. Some identifiable feature should be in the photograph to enable the photograph to be matched with the mapping photography. At least five exposures should be taken. Two cameras should be employed to provide backup.

The interval that will produce a 60% overlap can be computed from the formula:

$$i = \frac{1.44ah}{vf} \qquad (1\text{-}3)$$

where i = the interval in seconds between exposures

a = the length of the exposed portion of the film in millimetres (Vinten 57 mm, Hasselblad 51 mm

h = the flying height in metres above groundlevel

v = the velocity of the aircraft in km/hour

f = the focal length of the camera in millimetres. For 80% overlap change the constant from 1.44 to 0.72.

An extra picture should be taken at the end of the run because the intervalometer may not have completed its cycle when turned off and a double exposure can occur on the last picture. One-half metre of unexposed film should be left at the end of each exposed roll to allow for a stepwedge by the developer.

Pictures should not be taken at shutter speeds slower than 1/500 of a second from fixed-wing aircraft, and preferably at greater speeds from a helicopter because of the vibration. An exposure meter is recommended for cameras with manually-controlled diaphragms.

In a fixed-wing aircraft the pilot's view of the target is often obscured by the nose of the plane. If there are no reference points on line the plane may drift and it is possible to miss the target. One alternative is to fly a course at right angles to the intended photo flight direction. At 3 to 6 km from the target, the target will be visible from the side of the aircraft. When it is at right angles (abeam) make a right-angle turn in the direction of the target and hold the heading. Turn the camera on after the turn. (Pilots are very adept at flying "traffic pattern", that is, a series of right-angle turns.)

Camera lenses should be checked frequently for the presence of dirt, oil, or water and cleaned with lens tissue as required.

1.3.4 ASA Ratings and Processing of 70 mm Film

Initial Processing

Unless arrangements are made to do field-processing, the films from the two cameras will be forwarded to the Section Chief in Ottawa in separate shipments. Processing will be initiated and will follow one of three routes:

a) For 70 mm film, where a rapid service is necessary, the Section Chief may arrange processing to the negative stage through Mapping and Charting Establishment (MCE).

b) Normal processing of 70 mm film will be done through the National Air Photo Library using a standard requisition form.

c) Processing of 35 mm film will be done through the Canadian Government Photo Centre using their order form. One print of each negative, at contact size, is to be ordered.

Two types of 70 mm film are presently used by Geodetic Survey - Plus-X and Tri-X Aerographic Film (Estar Base) 2403. Unexposed film should be handled in total darkness; do not use any safe lights.

ASA rating for Plus-X films is 400, and 800 for Tri-X. These ratings apply if the film is processed under the following conditions.

Processing: Chemistry 885
No. of racks 2
Machine speed: 3.66 m/min. (12 feet per minute)
Developing Temperature: 29.4°C (85°F)

The film should be sent to the National Air Photo Library for processing. Each can of exposed film should be labelled showing the type of film, ASA rating used, the length of film on the roll and the processing chemistry to be used. A covering memo should accompany the film giving all particulars of the film, including batch number, ASA rating used, camera type, average flying height, length of exposed film, length of unexposed film and number of contact prints required. The information on the can is required as the accompanying memo does not always reach the processor.

1.3.5 Photography Records, Annotation, Transmission and Storage

Apppendix D, page D1-1 shows a sample field film log.

Annotation

The main identification photography (normally the 70mm) will have the following annotation added by the field section involved, in black drawing ink, to one negative of the stereo pair showing the target:

a) a 2 cm triangle centered over the target;

b) a north arrow;

c) the station number; and

d) the roll number, including the year and field officer's initial (i.e. 83-S-01 stands for roll 1 by Smith in 1983).

Auxiliary identification photography (i.e. 35 mm) will be annotated, as above, directly on the print. Vertical control points may only have oblique photographs of the station. The location of these stations should be transferred and pin-pricked on the 9 x 9 mapping photography.

Final Processing

Those frames on the 70 mm rolls that are to be used for identification purposes (stereo pair of each target), will be listed and returned to the National Air Photo Library (NAPL) for final processing, by the field section involved. Two contact prints and one film positive of each negative listed will be obtained. Rolls of film are not to be cut.

Transmission of Data to the Topographical Survey Division

All film positives, prints and negatives, index maps, aerial photographs, descriptions and film logs, will be filed with Data Services Section. A copy of the film log, film positive, aerial photographs and one copy of the prints, shall be forwarded immediately by Data Services Section to the Topographical Survey Division with a request for assessment.

Storage of Data

Once the identification photography has been returned from the Topographical Survey Division, Data Services Section will ensure its secure storage.

a) One copy of each print and film positive of horizontal control points will be stored in hard copy (Monument Record Cards).

b) Negative rolls of 70 mm and 35 mm photographs will be stored in a climate-controlled fire resistant vault.

c) The 9 x 9 aerial photographs, oblique photos or other prints with control points identified on them, will be stored with the Data Services Section.

d) In cases where the only identification of a vertical point (altimeter traverses, etc.) is the marking on a 9 x 9 mapping photo, these photos are to be micro-filmed and the negative stored in the climate-controlled fire resistant vault mentioned in (b)

2 *PLANNING AND PREPARATION*

2.1 Project Initiation

Requests for supplementary control surveys originate from federal and provincial governmental agencies and the private sector.

All requests are considered by the Director, the Assistant Director (Surveys) and the Section Chief, in relation to the Division's approved Long Term Activity Plan, the resources available, and the Division's mandate.

A project, upon approval, is assigned to a field officer who assumes responsibility for the planning and execution of the project.

2.2 Selection of Survey Methods

The accuracy and density of existing horizontal and vertical control is evaluated in relation to the accuracy required in the new survey, to ascertain if additional framework control will be needed. The choice of survey methods and instruments to be used are based on several factors, such as:

a) accuracy required;

b) terrain type;

c) control density needed;

d) economy;

e) usefulness of control beyond immediate requirement.

Once the survey methods and instruments have been selected, several other factors must be considered in the detailed planning of the project.

2.3 Office Preparation

2.3.1 Survey Planning

Careful attention to detail in planning will help to ensure that a field survey progresses smoothly and efficiently. Following are some of the steps which will assist in planning:

a) Determine the number of survey points required and plot the approximate locations of these on a map showing the existing control.

b) Estimate the distances to be traversed by each of the various survey methods to be employed.

c) Order all maps, air photographs, station descriptions, coordinates, elevations, etc. that will be needed. Maps are supplied by the Canada Map Office, photographs by the National Air Photo Library and control data by the Data Services Section. Do not overlook control data that may be available from provincial and municipal sources.

d) Decide on markers best suited to the terrain and type of survey and determine the quantity required.

e) Determine the photo-identification and targetting needs of the survey.

f) Decide where auxiliary surveys will be required (e.g. azimuth determination, eccentric ties, etc.).

2.3.2 Personnel

Determine a) the number of people and the skills needed for the survey work and computations, and b) other personnel necessary for specialized and general operational support.

2.3.3 Transportation

a) Decide on the means of moving personnel and equipment.

b) Estimate the aircraft support and number of flying hours required.

c) Determine the size and locations of fuel caches.

d) Determine the number and types of ground vehicles.

2.3.4 Accommodation

The location of field headquarters will influence the selection and type of accommodation. Where no local accommodation is available, trailers or camping facilities must be provided.

2.3.5 Cost Estimates

A detailed cost estimate must be made of the survey (Form EMR. 11 Rev. Dec. 72) and submitted for approval. This estimate includes salaries and overtime for casual employees, accommodation and daily allowances where applicable, fuel costs, equipment maintenance, freight and shipping costs and all other operating costs for the survey.

2.3.6 Equipment, Supplies

a) Prepare a requisition form for all equipment to be supplied by Technical Field Support Services (TFSS). Specialized equipment not available at TFSS must be ordered well in advance of the date it is needed.

b) Obtain all necessary survey data recording forms, notebooks, tape cassettes and disks.

c) Complete and submit a field stationery list for all supplies.

2.3.7 Administration

a) Prepare contract specifications for aircraft requirements and submit these to the Department of Supply and Services (DSS) for tender.

b) Obtain all working permits required when working in the territories, national and provincial parks, and other restricted areas. Advance authorization may be required for aircraft flights crossing the Canada-U.S.A. border. This is normally handled by the air carrier. If the air carrier has a class 9 licence he does not require additional authorization.

c) Arrange for firearm permits, where appropriate.

d) Submit the final cost estimate for approval together with a request for a bank account, (refer to General Instructions for Field Parties manual).

e) Prepare a projected work progress schedule.

f) Prepare a logistics schedule for the various phases of the operation.

g) Arrange travel schedules, airline reservations and motel accommodation for personnel.

h) Inspect all technical, scientific or survey equipment and arrange for all necessary testing and calibration before shipping.

3 *FIELD SURVEY OBSERVATIONS*

3.1 **Introduction**

Instrumentation and observing procedures are usually chosen to eliminate or minimize the effects of random and systematic errors in observations. The competent observer must make every effort to ensure that blunders are detected. This requires some form of observational redundancy. Observation repetition does not ensure redundancy nor does agreement between repeated observations indicate the presence or absence of a blunder. For example, repeated measurements of an angle to a wrong target will not indicate the presence of this blunder.

The intent of this section is to stress the importance of applying independent checks on all observations so that a blunder does not go undetected.

3.2 **Blunder Detection**

3.2.1 Sources

A blunder may occur during the observing, recording or processing phases of a survey. Common blunders include:

a) observing

misidentification of one or more of the points involved, e.g. sighting the wrong target; or setting up on a reference marker or other nearby station.

mistakes in reading the instrument - for example, concentrating on the smallest divisions and misreading the whole units (degrees, metres, etc.).

instrumental - blunders can arise from improper use of the instruments. For example, improper levelling, reading the vertical circle instead of the horizontal, etc. Any deviation from the accepted procedures for use of a particular instrument may produce a blunder.

b) recording

incompleteness - lack of essential information. For example, vertical angles are measured to two stations which are at about the same elevation, but station identifiers are not included in the field notes. It may be impossible to decide which angle applies to which station.

data recording - a blunder may be caused by lack of clarity in notekeeping or by mis-recording observations (eg. transposing numbers). Notes, in addition to being accurate and complete, must be clear and legible.

c) data processing

interpretation - in processing field data it should not be necessary to interpret field notes. Recorded data must be legible and unambiguous. Too often various interpretations must be tried to find one which does not yield a blunder.

abstracting - a very common blunder in processing data is caused by incorrectly abstracting or copying field data. All data transfers should be checked independently and initialled.

d) original notes

all original field notes must be retained and filed. Any notes which are copied should be labelled as copies.

3.2.2 Observation Redundancy

Redundancy may be provided by the geometry of the survey or by independent observations. The geometry allows a computational check for a blunder, as in the angular misclosure of a triangle or the misclosure of a level loop. Observations provide blunder checks when they are independent of the observation being verified, e.g. a cross-line on an altimeter traverse gives an independent value for the common point elevation and hence a check on gross errors.

Often, the redundancy is obvious and is part of the survey; for example, on an EDM traverse, one would not think of not closing the traverse both in scale and orientation. Mapping control points or azimuth lines which are set out from the main traverse must be confirmed by an independent measurement (e.g. offset distance and angles).

Measuring a distance in both metric and imperial units (a feature possible with many EDM instruments) provides a limited blunder check on the distance. Of course, any marker misidentification or instrumental blunder may still go undetected.

The observer must introduce sufficient measurements to allow geometrical principles (triangle closure, etc.) to provide a computational check on observations.

3.2.3 Distance Measurement

Distance measurements include those made by a variety of EDM devices as well as by steel or invar tapes. When making distance measurements, the following precautions should be taken:

a) Measure the distance from both ends or re-measure the distance in two different units and compare results.

b) For taped distances, repeat the measurements using a different reference or zero mark or different observers.

c) For electro-optical measurements, the use of a 25 cm bar is recommended. This will give two measured distances which should differ by 25 cm thus providing an independent check.

d) A variation of the above (c) is to set the instrument eccentric at right angles to the line and remeasure all distances involved.

e) Never "dial in" zero error or met corrections; instead record the uncorrected slope distance and all other information such as meteorological readings, instrument heights, etc. which will be required later to correct the measurement. "Dialed-in" data usually cannot be verified.

Regardless of the care taken, a single measurement usually cannot be considered blunder-free until satisfactorily combined with other observations in some geometric model.

3.2.4 Angle Measurement

Angle measurements are relatively easy to keep blunder-free provided recommended observing procedures are followed. As with length measurements, blunders caused by occupying the wrong point or the improper use of the equipment can be avoided only through extreme care and attention.

When making angle measurements, the following precautions should be taken:

a) Ensure the theodolite is level and the plate level bubble is well-adjusted. The bubble should be checked before and after each measurement. It should never be re-levelled during a set. The set should be discarded, the instrument re-levelled, and the set remeasured if a dislevelment occurs.

b) Maintain a record of telescope collimation error by noting the difference between direct and reverse readings. One might spot a blunder by noting a drastic change in this difference.

c) Reduce angles on site. The measurements should be reduced to ensure there are no blunders present. Pay attention to the degrees and minutes as well as noting the spread in seconds.

d) When target identification is doubtful, take steps to rectify the problem. If there is doubt that the target light observed is the correct one, have someone flash the light on/off in a prescribed sequence. Make sure the notes include any doubts about the target. If the traverse closure indicates a blunder, this will give an indication of where to reobserve.

4 *VERTICAL CONTROL*

4.1 Levelling

4.1.1 Introduction

Lower-order levelling implies an accuracy of third-order or lower (Ref. S & M, 1978), and provides vertical control for mapping and other lower-order surveys.

4.1.2 Instrumentation

A standard surveying level is used, such as the ZEISS Ni2, WILD N2, KERN GK2A, etc. Both split-bubble types or those with automatic compensators are acceptable, but the latter has become more popular due to ease and speed of operation. Fixed-leg tripods are desirable, but not essential for lower-order work.

Rods should be graduated in metric units. Invar rods are not necessary. The rod is held plumb with the aid of a small hand-held rod level.

Portable metal turning points are used if stable points such as rock outcrops, cement curbs, etc. are unavailable. A bench mark chisel is necessary with bench marks having the shank installed horizontally. Adjustment pins for adjusting the level may be required.

4.1.3 Procedures

The starting bench mark is inspected for damage or possible movement, and the description updated if necessary. The instrument must be set up on stable ground. This is particularly important when using automatic levels because slow sinking of the level will be automatically compensated for and therefore may not be detected. Where ground conditions are unstable (swampy areas for example), two rodmen are required to avoid delay between backsight and foresight readings. It is essential that only stable turning points be used.

Foresight and backsight distances should be balanced - careful pacing is usually adequate. If three-wire levelling is used, the stadia measurements can be used to balance sights. Sight lengths should not be longer than one hundred (100) metres. The level must be checked periodically and adjusted if necessary. (Ref. 4.1.6 and Appendix C).

Temporary bench marks (TBMs) should be established to reduce the distance between bench marks to sections 1 to 1.5 km long. This allows individual sections to be re-levelled, rather than the entire line, when unacceptable misclosures occur. The TBM's should be stable objects such as rock outcrops, railroad ties and rails, bridge abutments, etc. and should be easily identifiable and clearly marked to facilitate recovery.

The Level line must have redundancy: i.e. it must either close on the initial bench mark or terminate on a second bench mark. One-way lines from a bench mark to an unknown point provide no check on the levelling and are unacceptable.

4.1.4 Recording Data

Level notes are to be recorded on form SCS 78-6. Appendix D (page D5-1) gives detailed instructions on using this form, as well as an example. Form SCS 78-6 provides a duplicate copy of field notes as they are recorded. This copy should be stored separately from the original for safe keeping.

4.1.5 Accuracies

The following lower-order specifications are used to determine whether a loop or line misclosure is acceptable:

Third-Order	$\pm$ 24 mm$\sqrt{K}$ or 0.10 ft$\sqrt{M}$
Fourth-Order	$\pm$ 120 mm$\sqrt{K}$ or 0.5 ft$\sqrt{M}$

where K (M) is the one-way distance between starting and closing bench marks in kilometres (miles). Sections having misclosures which exceed these limits must be re-levelled, and the suspect values rejected.

For more detailed information on levelling procedures, refer to (S & M, 1978) and (S & M, 1983).

4.1.6 Sources of Error

Successful levelling requires that instruments be in good adjustment and handled with care. The level must be checked at least once a week, or whenever it is suspected that damage may have occurred (i.e. from a fall). Simple field procedures can be carried out to adjust the level.

The major error sources in levelling can be minimized by proper field procedures. The following errors should be considered:

a) collimation error

This error occurs when the line of sight is not perpendicular to the plumb-line as defined by the level bubble or automatic compensator. The same error occurs in both backsight and foresight readings, and is self-cancelling when the two sight lengths are equal. Since sight balancing is approximate, it is essential that the level be well-adjusted. Appendix C gives detailed instructions on performing the "two-peg test" necessary to check and adjust the level collimation.

b) temperature effects

Whenever possible, the instrument should be shaded from direct sunlight to avoid errors resulting where one part of the instrument near one end of the bubble is at a higher temperature than the other. Automatic levels are less affected by direct sunlight.

c) sinking of rod and instrument

Because backsight and foresight readings are not taken simultaneously, the stability of rod and instrument is important. Care must be taken to select firm and stable ground for turning points and the tripod. Avoid excessive delays between backsight and foresight readings. This is particularly important when automatic levels are being used as the sinking may not be detected.

d) rod errors

In lower-order levelling, rod graduation errors and temperature expansion effects are usually not considered to be significant. The rod zero error is important only when two rods are being used. If the zero graduations of the rods differ, then care must be taken to avoid accumulation of this error. When two rods are used one rod should be used for the backsight at one setup and for the foresight on the next setup. If this pattern is repeated for an entire line, having an even number of setups, the accumulated error between bench marks will be minimized. The same rod should always be used on bench mark ties.

e) <u>non vertical rods</u>

This is a serious error even for lower-order levelling. For a rod reading of 2 metres and an inclination error of 3 degrees from the vertical, a reading error of 2.7 mm is introduced (Cooper, 1971). The error is easily eliminated by using a well-adjusted rod level. When one is not available, the rod should be swung slowly back and forth, towards and away from the instrument, and the lowest rod reading recorded. This reading corresponds to that which would be obtained if the rod were vertical.

f) <u>centering bubble</u>

The adjustment of the centering bubble is of particular importance for automatic levels. If it is significantly out of adjustment, the mislevelment error will exceed the range of the automatic compensator and result in erroneous readings. Adjustment of the centering bubble is simple. The bubble is centered using the levelling screws and the level rotated 180 degrees in the horizontal plane. The amount the bubble moves off-centre is twice the centering bubble error. The adjustment pin is used to move the bubble half-way back to centre, and the levelling screws are used to complete the centering. This procedure is repeated until the bubble does not move appreciably off-centre as the level is rotated through 180 degrees. The spherical rod-level bubble should also be checked and adjusted if necessary. To adjust this bubble, set up the level about 50 m from the rod. Turn the face of the rod 90° to the line-of-sight, centre the bubble and compare the verticality of the rod with the vertical line on the telescope reticle. If the rod is more than 0.2° (1 cm) off vertical correct for 1/2 the mislevelment using the adjusting screws. Check with the face of the rod perpendicular to the line-of-sight.

4.2 **Trigonometric Heighting**

4.2.1 Introduction

Trigonometric heighting is a method of determining the difference in height between two stations by observing zenith distances (or vertical angles), given the slope distance between them.

Fundamental height control is normally provided by levelling between bench marks. This is often not an economical method of providing lower-order vertical control for medium and small scale topographic mapping. Mapping control may be provided by trigonometric heighting or other vertical control techniques, e.g. altimetry, ISS. The method chosen depends primarily on the cost and the accuracy required. Trigonometric heighting has particular advantages for providing height differences along EDM traverses as it can readily be combined with horizontal control measurements.

4.2.2 Instrumentation

Zenith distances are measured using one-second reading theodolites (WILD T2A, KERN DKM2A, etc.). Some of these theodolites are equipped with automatic compensators that provide a correct reference for the vertical circle if the instrument is properly levelled.

Targets are normally lights, although measurements may be made to other types of targets such as: target sets, signal cloth, top of tower, cairn, range pole, etc. The observer should choose the target which is most easily bisected by the horizontal cross hair. He must ensure that the correct target height is measured and recorded.

A steel measuring tape is used to measure the instrument and target heights to an accuracy of 0.01 m.

4.2.3 Procedures

Simultaneous reciprocal zenith distance measurement is the most accurate trigonometric heighting technique. At any given time refraction conditions are usually the same at each end of a relatively short line (less than 10 km); therefore, the coefficient of refraction will be about the same at both stations. The curvature and refraction corrections will tend to cancel each other when the forward and backward zenith distances are meaned. The only corrections required for simultaneous reciprocal zenith distances are instrument and target height corrections.

Strict attention must be paid to observing reciprocal angles simultaneously. Refraction may change rapidly, so observations should be completed at both ends of the line within the same one-minute period. At least four sets of angles are to be read. A set consists of readings taken in both the direct and reverse positions. Radio communication should be used for the synchronization of the readings.

Observations should not be taken during unstable atmospheric conditions, e.g. dawn or dusk. Figure 4-1 represents typical fluctuations in the coefficient of refraction (k) in a 24-hour period.

The most stable conditions are likely to occur just after mid-day or the middle of the night.

The quality of results also depends on the length of line and the topography. The accuracy of the observations decreases as the length of line increases, but generally results equivalent to fourth-order spirit heighting are obtainable on lines less than 15 km. Because atmospheric refraction varies considerably close to the ground, grazing lines of sight must be avoided. This may require shorter lines or the use of towers.

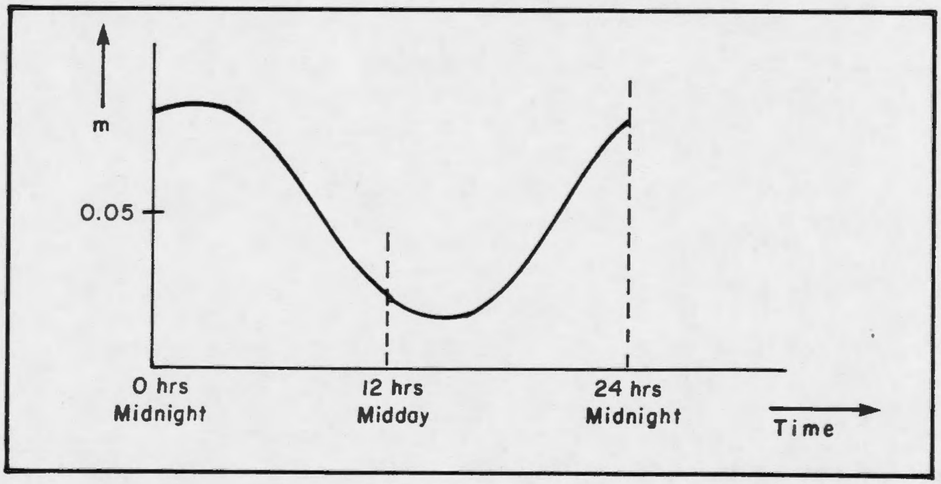


Figure 4-1 - Coefficient of Refraction Fluctuations

The following procedures for observing simultaneous reciprocal zenith distances are recommended:

a) Both observers ensure that their lights are pointed correctly, their instruments are levelled and target and instrument heights recorded before observations begin. When the observers are ready, the signal to begin observations will be given by radio, or by other means when radio communication is not possible. Lights should be masked to provide the smallest visible target.

b) Observations normally consist of four sets of direct and reverse pointings. At each pointing, the vertical circle bubble will be checked (for instruments without automatic compensators), the angle read, and the value and time of the observation recorded in the field book (see Appendix D, p. D4-1, 2, 3). For each set of observations, the difference between the sum of the direct and reverse pointings and 360° (for 1" reading instruments) will provide a measure of the vertical collimation error. This should be consistent over the period of observations. A large change in collimation may indicate a pointing error or that the instrument is off-level.

c) Both observations in a set will be reduced to a mean zenith distance. The zenith distances for all four sets will normally fall within an 8 arc-second range. When this range is exceeded, or when the observer suspects observations of inferior quality, additional sets of observations should be made.

d) Both observers should complete observations within a minute in time of each other. When reobservations are required for one observer, they must be made by both observers to maintain simultaneity.

4.2.4 Recording

Sample booking forms are included in Appendix D, pages D4-1, D4-3 along with brief instructions regarding the use of these forms.

The daily observations are checked after returning to the base camp, and abstracts are made of the relevant information, including those values that are suspect. Entries to the abstract sheets and computations are to be checked and initialled.

4.2.5 Computations

4.2.5.1 Simultaneous Reciprocal Zenith Distances

Sample computation forms are included in Appendix D, page D6-1. Elevation differences are computed from observed zenith distances using the following formula:

Refer to Figure 4-2.

$$\Delta h = h_2 - h_1 \tag{4-1}$$

$$= D \sin \frac{(Z_2 - Z_1)}{2} + \frac{(O_1 + t_1)}{2} - \frac{(O_2 + t_2)}{2}$$

where h_1 = elevation of station 1 in metres

h_2 = elevation of station 2 in metres

D = slope distance between stations in metres

Z_1 = observed zenith distance from station 1 to station 2

Z_2 = observed zenith distance from station 2 to station 1

t_1 = height of telescope at station 1 in metres

t_2 = height of telescope at station 2 in metres

O_1 = height of target at station 1 in metres

O_2 = height of target at station 2 in metres

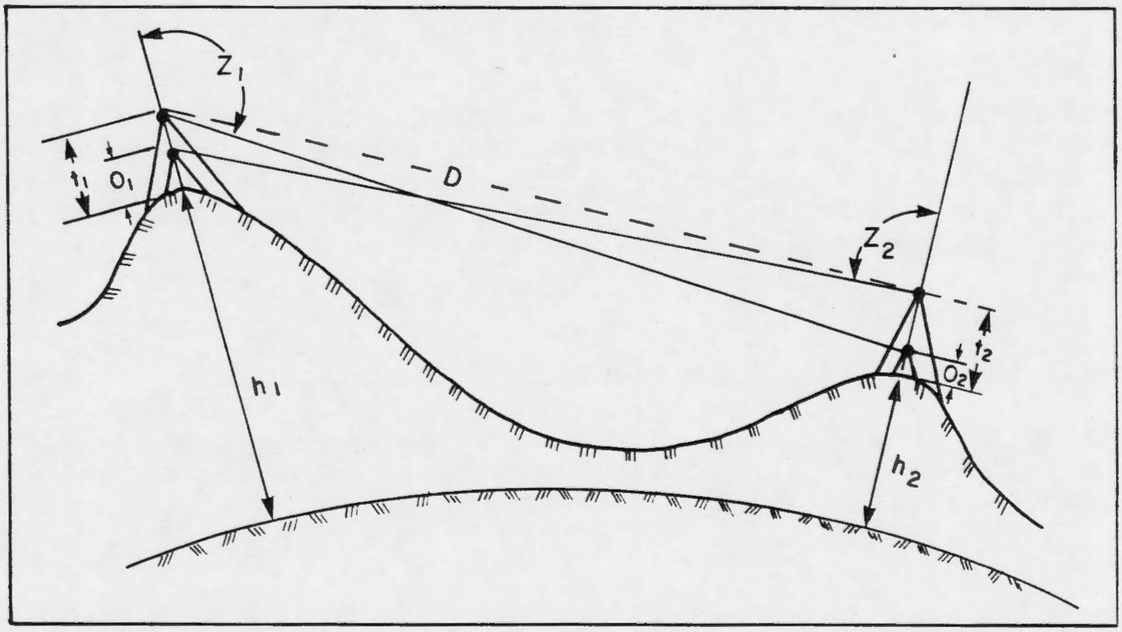


Figure 4-2 - Simultaneous Reciprocal Zenith Distances

The earth curvature and refraction corrections have been ignored because observations have been assumed to be simultaneous and reciprocal.

4.2.5.2 Non-reciprocal Zenith Distances

Non-reciprocal angles and non-simultaneous reciprocals are seldom used, due to poor accuracy and unreliability, therefore detailed computation procedures are not included here.

If non-reciprocal angles are measured, the two main corrections are for curvature and refraction. Computation procedures for all necessary corrections are detailed in many Surveying and Geodesy books to which readers are referred. (ex. Geodesy, Bomford, Third Edition, 1977).

4.2.6 Accuracy

Trigonometric heighting by simultaneous reciprocal vertical angles can meet fourth-order spirit heighting specifications (S & M, 1978, 0.38 m in 10 km). Line lengths should not exceed 15 km. The accuracy varies inversely with the line length and, for simple field adjustments on a closed figure, the misclosures can be distributed as a ratio of the line length to the total length of traverse.

4.2.7 Instrument Errors

There are essentially five instrument misadjustments that may be corrected by the observer:

1) Vertical axis not vertical.

2) Horizontal collimation error.

3) Inclination of the cross hairs.

4) Vertical circle index error.

5) Optical plummet off-line.

Only two of these, the inclination of the cross hairs and vertical circle index error, have any significant effect on vertical angle results.

4.2.7.1 Inclination of Cross Hairs

This problem can be eliminated during observations by always bisecting the target at the same point on the cross hair.

If after the telescope is transitted, the target is again bisected by the same point on the cross hair, the mean of direct and reverse pointings will be free of this error.

Although the foregoing observing technique will eliminate this error, if the inclination of the cross hair is noticeable, the following instrument adjustments should be made:

1) Level the instrument carefully.

2) Select a good target and bisect it with the vertical hair.

3) Using the vertical tangent screw, rotate the telescope about the transit horizontal axis. If the hair moves off the target, loosen the cross hair or reticle locking screws and rotate the reticle until the vertical hair stays on the target as the telescope is rotated about the transit horizontal axis. Tighten the locking screws.

4) Repeat the test until the adjustment is complete.

Note that if this adjustment is carried out, the horizontal collimation error will have to be checked, and adjusted if necessary.

4.2.7.2 Vertical Circle Index Error (Vertical collimation error)

Vertical collimation error is present if the vertical circle reading is not zero when the telescope is horizontal. A certain amount of collimation error is usually present and can be cancelled by observing on both direct and reverse pointings. Although in theory a large collimation error can be tolerated, in practice it should be reduced so that the surveyor can detect whether a gross error has occurred in his readings while observations are being carried out.

The usual adjustment method for theodolites with vertical circle bubbles is the following:

1) Level the instrument carefully.

2) Select a distant clear target. Bisect the target, carefully centre the vertical circle bubble, and read the vertical angle.

3) Repeat step 2 on the reverse pointing.

4) If there is no collimation error, the sum of the two readings will equal 360°, within normal observing errors. The difference between this sum and 360° represents approximately twice the collimation error.

5) Halve this value to estimate the actual error. Calculate the correct circle reading by adding or subtracting the collimation error and, with the telescope still on reverse pointing, set the proper circle reading using the micrometer and vertical circle levelling screw.

6) This will cause the vertical circle bubble to run off centre. Bring it back to its central position using the capstan headed adjusting screws.

7) Repeat the above procedures with different targets until the collimation error is less than 5" for a 1" reading theodolite.

The collimation error should be constantly monitored during observations as mentioned in step 4 above, and should remain relatively constant. This adjustment is not applicable to instruments with automatic compensators.

4.3 **Altimeter Traversing**

4.3.1 Introduction

Altimeter traversing is used to provide vertical control for mapping at 1:50 000 scale. This method of surveying provides relatively accurate and inexpensive control, provided that the procedures and techniques recommended in this section are followed.

4.3.2 Instrumentation

Geodetic Survey uses Wallace & Tiernan (W & T) altimeters for altimetric traversing. These are direct reading instruments, graduated in feet or metres and having an annular mirror so that parallax may be eliminated by superimposing the pointer on its reflected image when reading. The dial on each instrument is custom made. Negative readings are avoided by shifting the scale so that a reading of 1000 feet is obtained at sea level. The instrument is calibrated so that the altitude relates to pressure in accordance with Table No. 51 of the Fifth Revised Edition, Smithsonian Meteorological Tables (Smithsonian, 1939). The mechanism of the W & T altimeter is compensated for the effects of instrument temperature. For normal instrument temperature fluctuations, corrections need not be applied. A correction graph for each instrument is on the instrument cover.

A desiccant condition indicator on the altimeter dial turns pink when the desiccant needs replacing. If fresh desiccant is not available the old desiccant can be rejuvenated by drying at 150°C for ten minutes.

Three basic types of W & T altimeters having the same mechanism are models FA-181, FA-185, and FA-112. These differ only in the type of carrying case. They can be ordered to cover ranges of 4000, 7000 or 16 000 feet (type FA-112 is not available in the 4000-foot range). When an altimeter with a narrow range is required, an FA-176 with a range of 2000 feet is available. Altimeters with dials in metric units are also available.

When using helicopters for altimeter traversing, temperature probes with digital readouts - which indicate humidity as well as dry temperature - should be used rather than thermometers.

4.3.3 Calibration

Altimeters are calibrated once a year using either Paroscientific Model 230-A5-002 or Bell and Howell model 4-461-0003 digital barometers as a standard. The digital barometers are, in turn, calibrated by the National Research Council.

The 4000-foot range altimeters are calibrated at 200-foot intervals, 7000-foot altimeters at 500-foot intervals, and the 16000-foot altimeters at 1000-foot intervals. The altimeters are exercised in a pressure chamber and readings are taken as pressure is decreased, then exercised again and read as pressure is increased. The readings are recorded on "Altimeter Comparisons" form No. 17-2-78 and corrections and correction graphs are obtained using the Hewlett-Packard (HP)-87 computer and HP-7470A printer. Correction graphs are plotted as an output from the computer.

Before shipping, altimeters should be inspected for possible air leaks around the gasket under the window, and any leak found must be sealed. In addition, the rubber vent cap on the cover must be closed if the altimeter is to be transported at altitudes above its dial range. On arrival at their destination, altimeters should be inspected for damage, and should be compared in groups of 3 or 4 by connecting their vents to a manifold, reducing pressure by means of a hand pump, and comparing their readings at successive changes in pressure.

4.3.4 Principles

W & T altimeters used by Geodetic Survey are graduated in feet and are calibrated with respect to Smithsonian Meteorological Table 51 for an altitude-pressure relationship from the following formula: (Smithsonian, 1939)

$$Z = 62583.69 \frac{Tms}{283} \log \frac{P_0}{P} \tag{4-2}$$

where

Z = pressure altitude in feet

P_0 = Pressure of standard atmosphere (29.9" of mercury or 1012.53 millibars)

P = pressure at any level, same unit as P_0

Tms = mean temperature of air column between P and P_0, in degrees Kelvin for a standard temperature of 10° C

Tms = 273°K + 10°C = 283°K, and

$\frac{Tms}{283} = 1$

Equation (4-2) is applicable to a column of dry air with a temperature of 10°C. Corrections must be made for any variation of the density of the air due to temperature and relative humidity (vapour pressure). The small correction due to changes in the gravity field may be ignored.

Temperature correction to be applied is obtained from (Brombacher, 1944):

$$Tc = 3.5406\ (Tm-10) \frac{dh}{1000} \tag{4-3}$$

where

Tc = temperature correction in feet

Tm = mean temperature of the air column in °C

dh = difference in height measured by the altimeter between the two stations observed

With an increase of water vapour, the air becomes lighter than dry air and the correction for humidity is (Brombacher, 1944):

$$C_h = 0.376 \left(\frac{e}{p}\right)_m H_c \tag{4-4}$$

where

C_h = correction for humidity in feet

$\left(\frac{e}{p}\right)_m$ = the mean value of the ratio of water vapour pressure to the corresponding air pressure.

H_c = the height of the air column corrected for temperature (dh + Tc) in feet.

When the difference in height along an altimeter traverse is less than 150 metres, this correction is usually ignored.

Altimeter readings taken inside a helicopter are also affected by changes in cabin pressure, caused by changes to the heater or vent control settings, changes in the helicopter blade pitch and in the wind velocity. It is common practice to land the helicopter facing into the wind.

Atmospheric pressure is not stable but varies with meteorological changes. In a low pressure air mass, the wind circulates counter-clockwise; in a high pressure mass, clockwise; and the wind direction near the ground is tangent to the isobars. The size of an air mass can vary from several hundred miles to a few miles in diameter and its velocity can vary from zero to about thirty or so miles per hour.

Figure 4-3 shows an area covered by a low pressure air mass where three-altimeter traverses are considered. Even though the traverse C-G would have a high residual correction, one would expect it to give good results. The residual correction for traverse C-D would be very small, but the elevations in the middle of the traverse would be inaccurate. Good results would be obtained from traverse C-K.

The wind velocity at each traverse station should be recorded. This will give an indication of the position of the isobars.

When facing into the wind the rate of change of pressure will increase on the left by an amount that can be calculated from the formula (Crone, 1961):

$$G = \frac{V \sin \phi}{403} \qquad (4\text{-}5)$$

where

G = pressure gradient in millibars per mile

V = wind velocity in m.p.h.

ϕ = latitude

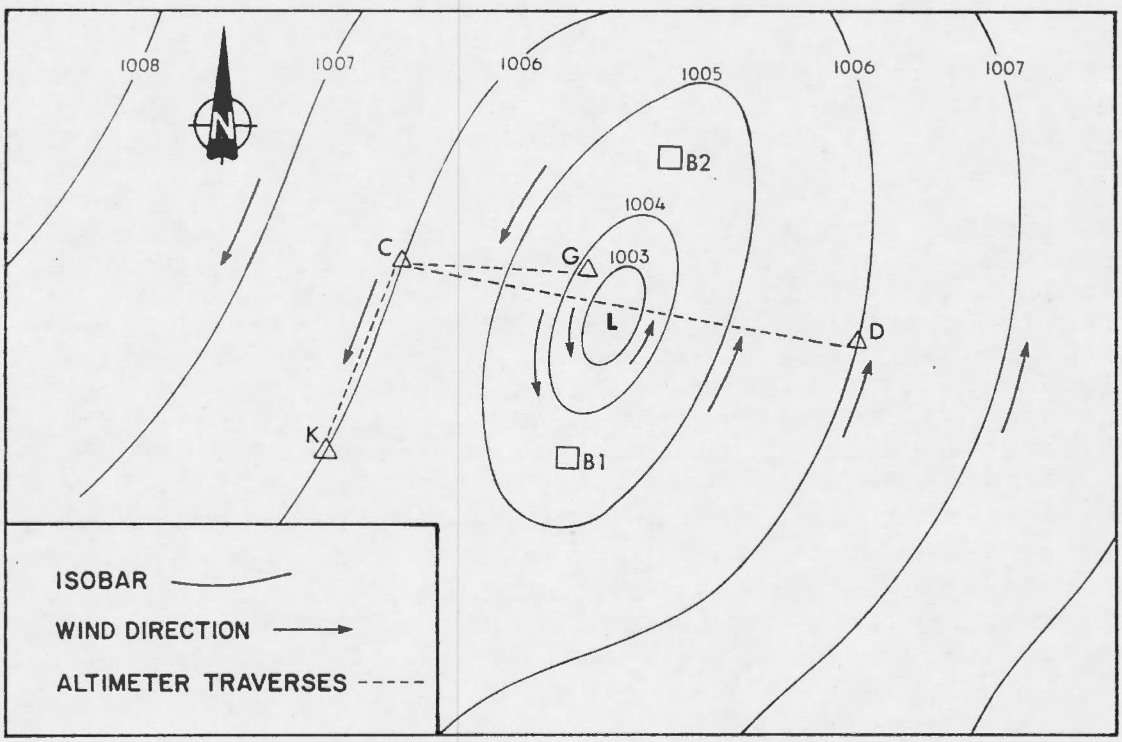


Figure 4-3 - Cyclonic Circulation in Northern Hemisphere

For example, let us take traverse C-G in Figure 4-3 and assume the following:

Latitude = 60°
Traverse length = 36 miles
Wind velocity = 30 MPH
Traverse is at right angles to the isobars (maximum correction)

$$G = \frac{30 \times 0.866}{403} = 0.064 \text{ mb/mile}$$

For a 36-mile traverse, the pressure difference is

$$0.064 \times 36 = 2.3 \text{ mb}$$

A millibar is approximately equal to 30'. Therefore one could expect a residual correction on this traverse of 2.3 x 30 = 69 feet.

Formula (4-5) is approximate and, though not normally used in the calculation of an altimeter traverse, is useful in determining the magnitude of the residual correction to be expected.

The movement of the air mass can be detected by recording the readings of a bank of altimeters (minimum 2) at a base station. The change in these readings with time is used to compute a gradient correction for an altimeter traverse. When gradient corrections are applied to the roving altimeter readings, it is assumed that the pressure varies throughout the area in the same manner as at the base station. This is seldom the case. If the air mass represented in Figure 4-3 moved in a southerly direction, the pressure at base B_1 would be decreasing while increasing at B_2. Applying the gradient corrections from B_2 to line C-G would be incorrect. Therefore, unless the base station is in the immediate area of the traverse, it is better not to apply the gradient correction in the computation; it will be corrected for in the residual correction.

The elapsed flying time between altimeter stations is usually less than five minutes, and between control less than one hour. In one hour, the gradient correction rarely exceeds fifteen feet. Gradient correction graphs should always be plotted, whether the corrections are applied or not, as they indicate pressure fluctuations which, if too erratic, may result in the traverse having to be rerun.

4.3.5 Procedures

The Geodetic Survey Division employs a technique called the "Doucette Modified Topographical Method" to compute elevations by altimetry. The traverse starts at a known elevation, and the difference in height between the known and first traverse point is obtained by differencing the altimeter readings at each point. This difference, corrected for variation in the density of the air column (temperature correction), is then added to the known elevation to derive the elevation of the first traverse point. This point, in turn, is considered to be one of known elevation and the traverse continues. The last point in the traverse is one of known elevation and the difference between the computed value and the known value, called the "residual correction", is distributed back through the traverse as a function of distance from the starting point divided by the total traverse distance.
A helicopter is used for transportation. Suitable control elevations and fuel stations must be available or established before the altimeter traverse is run. The progress of an altimeter traverse should be uninterrupted once begun. A traverse should be run in a fairly straight line between points of known elevation. Time, temperature, humidity and altimeter readings are recorded at each point including control points.

The traverses are run in one direction on one day and in the reverse direction on a different day. Identifiable features (lakes) are used for traverse points or temporary markers are left to ensure that the same points are used on each run.

The following procedures are recommended:

a) As an aid to identification on the mapping photography, take a Polaroid picture on approach about 0.5 to 1.0 km from the traverse point.

b) Read and record air temperature and humidity before landing to avoid the effect of surface heat radiation on the thermometers. Readings of digital meteorological equipment (temperature °C and humidity) are to be taken after flying at about 40 metres above ground level for a period of about 1 minute. The meteorological equipment needs a flow of air through it for accurate temperature and humidity observations.

c) Land on the point where an elevation is required facing into the wind.

d) Have the pilot reduce the blade pitch to zero and maintain a constant rotor R.P.M.

e) After about 30 seconds, read and record the three altimeters and the time of the reading. Note the approximate wind speed and direction.

f) Identify the point on the mapping photo and on the Polaroid picture:

g) Leave a marker on the ground to ensure recovery of the point on subsequent traverse runs, if lakes are not used for traverse points.

h) On leaving the station, take an additional Polaroid picture if the original picture was not satisfactory.

i) Proceed to the next station.

j) Read and record altimeters twice on all control stations, with a separation of about one minute between readings.

4.3.6 Major Factors Affecting Quality of Altimeter Traversing

a) Altimeters

Three altimeters should be observed when traversing. These altimeters should be selected on the basis of calibration graphs, those with the most linear and constant correction being favoured. Only altimeters whose pointers remain steady under the vibrating conditions in a helicopter should be used. In the helicopter, always take readings under similar conditions, i.e. no pitch on rotor blades and approximately the same rotor R.P.M. This maintains the same helicopter induced pressure at each station. Tests should be made to determine the difference between readings taken in the helicopter and those taken at ground level with no effect from the helicopter blades. This difference may be needed when reading outside the helicopter on a bench mark or trig. station (see Helicopter/Ground Corrections, section 4.3.6 h).

b) Pressure Gradient

Atmospheric pressure at any point varies with time. A graph can be plotted of base barometer readings versus time. If this graph is a straight line or nearly so, traverse results should be good. If it is too erratic, the traverse should be rerun under more stable weather conditions.

c) Weather

Make general notes on weather conditions. If the wind direction suddenly changes by an appreciable amount, it indicates that atmospheric conditions are unsuitable for altimeter traversing.

d) Time

Delays must be avoided on altimeter traverses to lessen the effects of large variations in atmospheric pressure. If it is necessary to stop for fuel, lunch, etc., always make such stops at a point of known elevation. Take readings when landing and again before leaving.

e) Temperature

On a sunny day temperatures observed close to the ground are usually not representative and will give a false correction in traverse computations. The temperature required is that of the free air without local heating or cooling effects.

f) Selection of Traverse Routes

Where possible, traverse routes should follow a straight line and the new points selected should all be at approximately the same elevation. Ideally, the elevation difference between any two traverse points should be less than 300 m.

g) Point Selection and Identification

Altimeter points are preselected by careful stereoscopic inspection of the mapping photography to ensure the choice of stereoscopically-identifiable points. A 2.5 cm diameter circle is used to indicate the position of proposed points on the mapping photography. Once occupied, points must be accurately marked on the photographs. This is usually done by a small pinprick. If a point selected is the highest or lowest point of a feature, then the point should be marked with a circle rather than a pinprick on the mapping photograph. The plotter operator can identify the highest point of a feature stereoscopically, but it is important that a notation "the highest point" refers, in fact, to the highest point. A hand level should be used to resolve any doubt.

h) Helicopter/Ground Corrections

Normally all observations are made from within the helicopter. When it is not possible to land directly on a station, the usual practice is to land nearby and read the altimeters sitting on the station mark. It is then necessary to apply a correction to make these readings consistent with those taken in the helicopter. This correction will be dependent on the type of helicopter, the rotor blade conditions at observations (R.P.M. and pitch) and the location of the altimeters within the helicopter.

It has been found that calibrations for some Hughes 500D helicopters are not consistent, and vary unacceptably.

To determine the difference in readings taken inside the helicopter and those away from the influence of the helicopter blades, readings of three altimeters are taken in the helicopter with no blade pitch and a constant R.P.M. and again outside the helicopter about 20 metres away. A minimum of 10 sets of readings are observed. This correction should be determined for each individual helicopter.

4.3.7 Recording

Form SCS85-1R (see Appendix D, page D7-1) is used to list the altimeter traverse data for input to an HP-9830 or Apple IIe computer for computation.

A brief description of the station should be entered on the form to help clarify the station's position, for example, water level, highest point of hill, low point of saddle, etc. Also, any change of weather, such as cloud cover, wind direction and strength should be noted. The wind direction and velocity can be shown on the form at the top of the page. Any changes in direction or velocity should be included in the notes column of form SCS85-1R.

After completion of the day's traversing, the "Lawnikanis North" and "East" columns are completed. The readings are taken from the "Lawnikanis Photo Pin Pointer Mark II" to facilitate locating the station to the nearest millimetre on a copy of the aerial photo, should the original be lost. Position the transparency with the lettered side on the same edge as the photograph number. Align the heavy centre lines with the fiducial marks and centre the transparency on the photograph. Read and record the grid coordinates to the nearest millimetre. (Estimate to the nearest tenth of the centimetre grid). For example, a station located in the centre of the lower square would be recorded as 01.5 x A0.5. The headings "North" and "East" refer to the Pin Pointer - not to the actual direction of north and east.

4.3.8 Computations - Doucette Method

Altimeter traverses begin and end on points of known elevation. The first point is used to determine the elevation of the second, which in turn is used to determine the elevation of the third, and the traverse progresses to the final point of known elevation. The closing error (residual) is then distributed back through the computed elevations.

Final elevations are computed from:

$$Elev_i = Elev_{i-1} + (Alt_i - Alt_{i-1}) + TC_i + HC_i \ldots + RC_i \quad (4\text{-}6)$$

where

$Elev_i$ = elevation of any station (feet)

$Elev_{i-1}$ = elevation of previous station (feet)

Alt_i = mean altimeter reading at station i (feet)

Alt_{i-1} = mean altimeter reading at previous station(feet)

TC_i = temperature correction (°C)

$$= 3.5406 \times 10^{-3} \frac{(Temp_i + Temp_{i-1})}{2} - 10 \quad (Alt_i - Alt_{i-1})$$

(Smithsonian, 1939, Table 52)

HC_i = humidity correction

$$= 0.376 \frac{e}{p} (Alt_i - Alt_{i-1})$$ (Brombacher, 1944)

where e is the mean pressure of vapour in the air column and

p is the mean barometric pressure

RC_i = residual correction at station i

$$= \text{Total residual correction} \times \frac{d_i}{\text{Total } d}$$

where d_i is the distance from the starting control point to station; and

d is the distance between control stations

The Geodetic Survey Division computes altimeter traverses on a number of programmable calculators or micro-computers (HP9830A, Apple IIe, TI-59).

4.3.9 Accuracy

Elevations by altimetric traverse may be obtained with a standard deviation of at least 1.5 metres provided that:

a) a battery of three precise altimeters is read at each station. Only altimeters having the most consistent calibration graphs are selected for use;

b) the elevation difference between the lowest and highest stations in the traverse is less than 300 metres and temperature corrections are applied;

c) the traverse stations are in a relatively straight line between control stations - the corridor should not be wider than 0.1 times the distance between the control points;

d) the distance between control points is less than 60 kilometres. This can be extended to 100 kilometres if altimetric tie lines are run across the middle of the traverse to provide additional control;

e) the traverses are run on two different days, and the computed elevation difference obtained for any point does not exceed 3.0 metres;

f) the weather conditions are stable, the wind steady and less than 30 kilometres per hour;

g) the traverses are run rapidly without interruption at a steady rate of progress of about 70 kilometres per hour;

h) all altimeter traverses have at least one external check station.

A blunder in reading one of the altimeters will be flagged in the computer output. A misreading of all three altimeters at a station, by the same amount, will go undetected unless the elevation of the station is determined again by running the traverse a second time.

4.4 Inertial Surveying

Since 1975 much of the control needed for mapping, especially south of latitude 60°, has been provided by inertial surveying. An Inertial Survey System (ISS) is regarded as a total survey system that can provide three dimensional control - latitude, longitude and elevation. In addition, its potential to provide deflection and gravity data has been demonstrated.

Elevations determined by inertial surveying are usually classed as equivalent to fourth-order spirit levelling. In general, elevations provided by ISS are considered to have an accuracy better than 50 cm relative to control, where traverses are 80 to 100 km long, stations are spaced 8 to 10 km apart, zero updates (ZUPT) are at 4 minute intervals, and mode of transport is a helicopter.

Inertial surveying is described in more detail in Chapter 5.3.

4.5 Ground Elevation Meter

The Ground Elevation Meter (GEM) consists of a four-wheel drive truck with four wheel steering capability. A fifth wheel houses a velocity pulse generator (a pendulum device which measures changes in slope angle) and a computer which accepts the input inclination and velocity signals and computes relative elevations.

The GEM has been used by Geodetic Survey since 1975 to provide vertical control for mapping.

The survey procedures involve four basic steps: planning, reconnaissance, calibration and traversing. A traverse consists of nulling the system at a starting control point and driving to, and recording data at each pre-arranged point, including the terminal control point. Traverses are kept under 50 kilometres in length. At least one check point of known elevation is required on each traverse.

The misclosure at a terminal bench mark is considered to be caused by systematic errors (wheel slippage and pendulum drift) and is distributed through the traverse as a linear function of distance. The mean of forward and back adjusted elevation determinations represents the final field value for each new point.

Experience at the Geodetic Survey of Canada indicates that elevations, with a relative accuracy of 50 centimetres or less, can be achieved using current procedures.

Because this survey method is becoming obsolete details are not included in this manual.

4.6 **Doppler Satellite Heighting**

Doppler processing software computes positions in an X, Y, Z coordinate system or in a latitude, longitude and height ellipsoidal coordinate system.

The height is always the height of the survey marker above the reference ellipsoid. Usually the processing software allows the user to specify the desired ellipsoid by the semi-minor and semi-major axes and the position of its origin in relation to the earth's centre of gravity.

It is generally accepted that this ellipsoid height is as accurate as the horizontal position.

To obtain the Doppler-derived orthometric height (elevation above mean sea level) the appropriate geoid-ellipsoid separation must be applied. In general, the resultant accuracy of the Doppler-derived orthometric height is a function of:

a) the accuracy of the applied geoid separation at each site,

b) the accuracy of the known orthometric heights at the Doppler base stations,

c) the location of each new station in relation to the base stations,

d) the Doppler positioning techniques used.

Doppler positioning is described in more detail in Section 5.2.

4.7 **Sea Level Referencing**

4.7.1 Introduction

When vertical control is to be tied to sea level, the time and date of the observation must be recorded. Using tidal information, the height of the sea surface with respect to mean sea level may be computed and used to control a vertical survey.

Mention is made here of this aspect of heighting because it is used extensively in Northern Canada, particularly in the Arctic Islands where other forms of vertical control are absent.

4.7.2 Technical Information

Fluctuations in the height of the sea surface are mainly periodic and predictable. Therefore, height at a specified time can be predicted. There are four major tide types:

a) Semi-diurnal tide - two complete oscillations daily with the two high water levels (and also the two low water levels) having similar heights;

b) Mixed, mainly semi-diurnal tide - two complete tidal oscillations daily, but with unequal tidal heights and irregular intervals between extreme levels;

c) Mixed, mainly diurnal tide - usually two complete oscillations daily, but with significant inequality in height and time;

d) Diurnal tide - one complete oscillation daily.

Generally, tidal height predictions are based on long term (usually in excess of one year) observations of the sea level at tide gauges. The accuracy of prediction is dependent on how far from the tide gauge an observation is made. Tidal characteristics a few kilometres along the coast from a tide gauge may be quite different from those at the gauge.

Predictions are made using tables published under the authority of the Canadian Hydrographic Service. These are published annually, as "Canadian Tide and Current Tables", in six volumes. Each volume relates to specific geographical limits (Figure 4-4). In addition, the publication "Water Levels - Tidal Highs and Lows" is published in one volume, usually two years after tidal observations are taken, and gives observed tidal heights. It can be used to estimate the accuracy of the predictions mentioned above. For areas of the arctic not listed in the tide tables information may be available from Marine Environment Services, 200 Kent Street, Ottawa, Ontario, K1A 0E6.

The following basic definitions should be noted:

Reference ports: ports for which predictions are made in the form of daily tables of times and corresponding water heights.

Secondary ports: ports for which differences in times and heights, with respect to a reference port, are predicted.

Differences: the adjustments which are applied to the predictions at reference ports to obtain predictions at secondary ports.

Datum: datum for predictions is Chart Datum. The Canadian Hydrographic Service has adopted the plane of lowest normal large tides, and tidal heights refer to this datum. Mean sea level elevations can be obtained by subtracting the height of mean water level from the computed height of tide.

4.7.3 Prediction of Tide Height and Elevation

Predicted tide levels for reference and secondary ports at regular time intervals are published. Levels for intermediate times may be interpolated using a simple cosine function.

NOTE: Time given in reference and secondary ports tables is standard time for the geographical area of the port.

4.7.3.1 Prediction of Tide Height at Reference Port
(Refer to Figure 4-5)

Given t_0 = standard time of observation

Determine h_0 = height of tide at Reference Port at time t_0

Where t_1 = time from tables immediately before t_0

t_2 = time from tables immediately after t_0

h_1 = height of tide at t_1 from tables

h_2 = height of tide at t_2 from tables

x = range of tide divided by two = $\frac{h2 - h1}{2}$

$$\alpha = \frac{t_0 - t_1}{t_2 - t_1} 180$$

$$Y = x \cos \alpha$$

$$x = h_0 - h_1 + Y$$

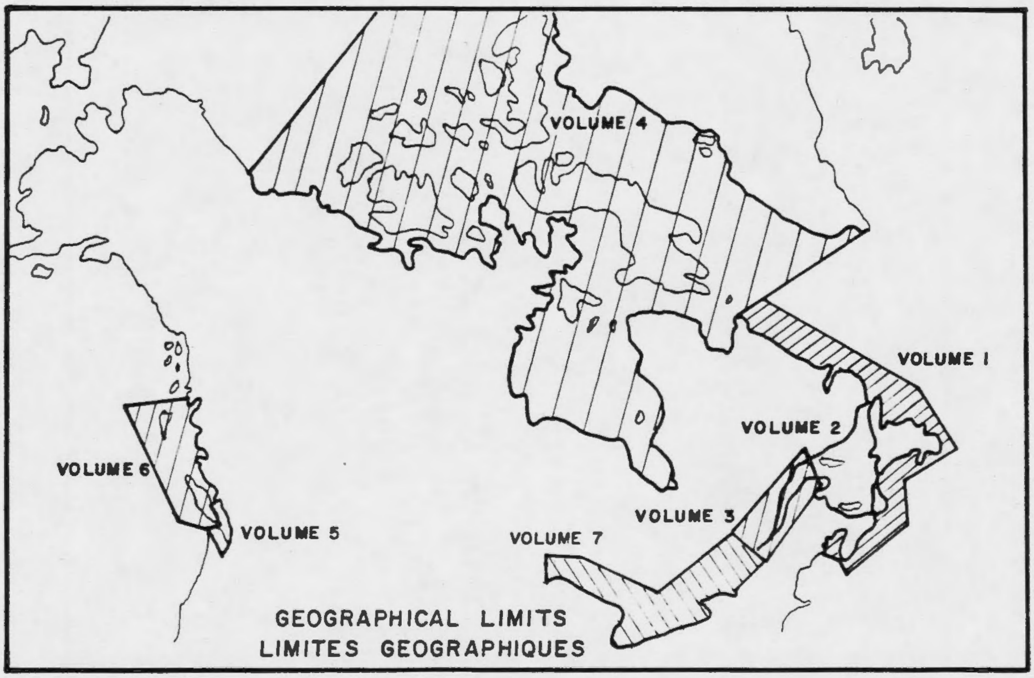


Figure 4-4 - Canadian Tide and Current Tables

Therefore,

$$h_0 - h_1 + x \cos \alpha = x$$

$$h_0 = h_1 + x\,(1 - \cos \alpha)$$

$$h0 = h1 + \frac{(h2 - h1)}{2}\,(1 - \cos \alpha) \qquad (4\text{-}7)$$

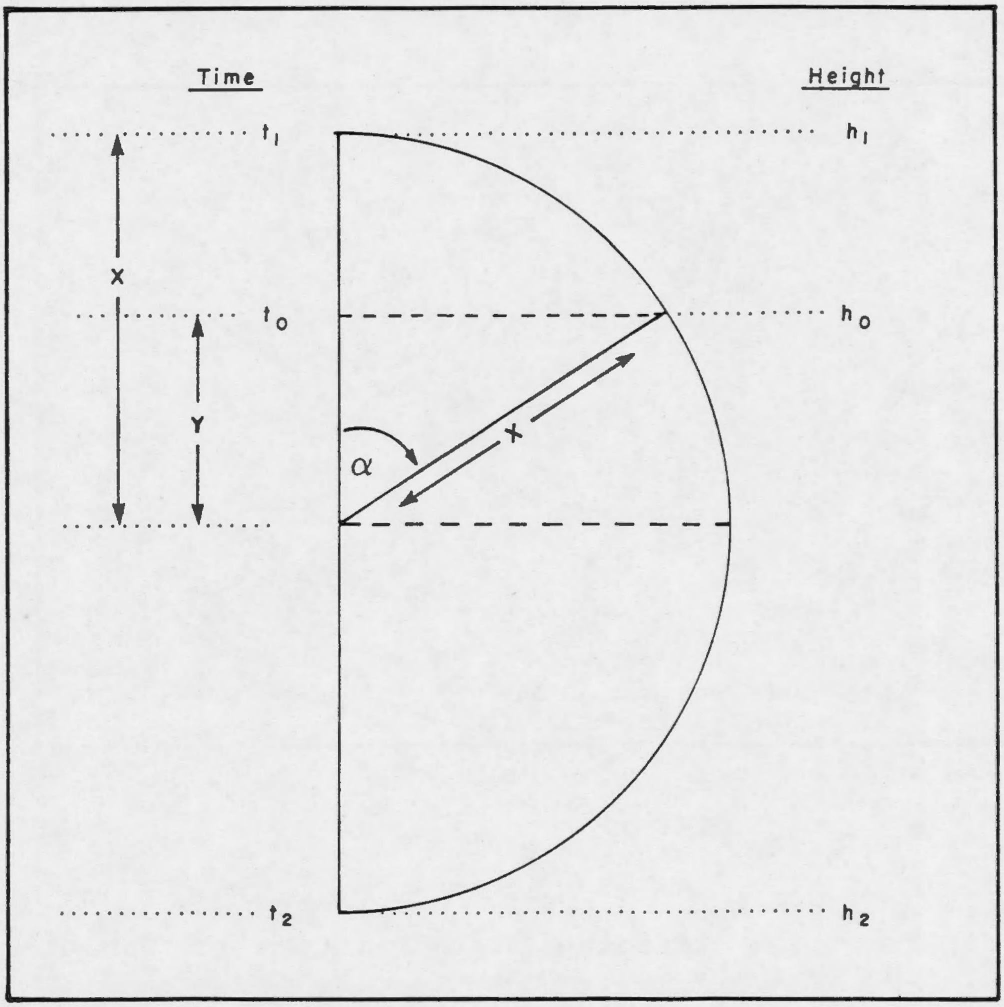


Figure 4-5 - Interpolation of Tide Levels

For example, an observation of water level was made at Bay Head on July 2 at 18:20 hours (daylight saving time). Using the tide tables, the following data is obtained:

$$t_0 = 18{:}20 - 1{:}00 = 17{:}20 \text{ standard time}$$

$$t_1 = 16{:}00, \quad h_1 = 0.24 \text{ m}$$

$$t_2 = 22{:}30, \quad h_2 = 4.51 \text{ m}$$

$$\alpha = \frac{17{:}20 - 16{:}00}{22{:}30 - 16{:}00} \times 180^\circ = 36^\circ.923$$

$$h0 = 0.24 + \left(\frac{4.51 - 0.24}{2}\right) (1 - \cos 36^\circ.923)$$

$h_0 = 0.67$ m

NOTE: This is the height of tide at t_0 with respect to chart datum. From tables, it is found that the mean water level at Bay Head is 1.7 metres. Therefore, the approximate height of tide with respect to mean sea level is 0.67 m - 1.7m, or -1.03 m.

4.7.3.2 Prediction of Tide Height Using a Secondary Port

a) Given the date and the standard time of observation of water level, determine the nearest secondary port.

b) From tables, abstract the following information concerning the secondary port:

E = time difference, higher water
F = height of mean tide, higher water
G = height of large tide, higher water
H = time difference, lower water
I = mean tide, lower water
J = large tide, lower water
K = mean water level

NOTE the corresponding reference port.

c) From tables, abstract the following data concerning the reference port:

A = high water mean tide
B = high water large tide
C = low water mean tide
D = low water large tide
L = FROM time (time immediately preceding the time of observation from daily tables)
M = tide height at time L
N = TO time
θ = tide height at time N

NOTE: When abstracting the information, the time differences from the secondary port for the high and low tides must be added to the respective high and low tide times of the reference port. Check whether the time of observation falls within these corrected times. If it does not, the next (either earlier or later) TO and FROM times and tides must be used to obtain the correct result.

d) If $M > \theta$ (FROM high tide TO low tide)

Compute T:

$$T = \frac{(M - A)\ (G - F)}{B - A} + F + M$$

NOTE: IF A = B, THEN $T = \frac{F + G}{2} + M$

Compute U:

$$U = \frac{(\theta - C)\ \ (J - I)}{D - C} + I + \theta$$

NOTE: If D = C, then $U = \frac{I + J}{2} + \theta$

$V = L + E$, where V is corrected time for high water

$W = N + H$, where W is corrected time for low water

If $M \leq \theta$, (FROM low tide TO high tide)

Compute T:

$$T = \frac{(M - C)\ (J - I)}{D - C} + I + M$$

NOTE: If D = C, then $T = \frac{I + J}{2} + M$

Compute U:

$$U = \frac{(\theta - A)\ (G - F)}{B - A} + F + \theta$$

NOTE: If A = B, then $U = \frac{F + G}{2} + \theta$.

$V = L + H$, where V is corrected time for low water

W = N + E, where W is corrected time for high water

e) Compute sea surface height with respect to mean sea level at time of observation, Z.

$$h = \cos\left[180^\circ \times \frac{(Z - V)}{(W - V)}\right] \times \frac{(T - U)}{2} + \frac{(T + U)}{2} - K \quad (4\text{-}8)$$

NOTE: All times must be expressed in hours and decimals of hours. Strict attention must be paid to algebraic signs.

Example of Sea Level Tie at Chesterfield Inlet

Given: Observation to sea surface on August 19, 1976 at 14:40 daylight saving time, Z = standard time = 14:40 - 1:00 = 13:40 standard time.

a) The secondary port is Chesterfield Inlet, the reference port is Churchill.

b) From tables, data for Chesterfield Inlet (secondary port):

E = -3:47 = -3.7833 , F = -0.30 m , G = -0.50 m

H = -3:48 = -3.8 , I = 0.20 m , J = 0.50 m

K = 2.40

c) From tables, data for Churchill (reference port):

A = 4.10 m B = 4.60 m C = 0.70 m

D = 0.00 m L = 13:55 = 13.916 hours

M = 3.5 m N = 19:45 = 19.75 hours

θ = 1.7 m

d) Since M > θ, then observation is FROM high tide TO low tide, and

$$T = \frac{(3.5 - 4.10)(-0.5 - (-0.3))}{(4.6 - 4.1)} + (-0.3) + 3.5 = 3.44$$

$$U = \frac{(1.7 - 0.7)\ (0.5 - 0.2)}{(0 - 0.7)} + 0.2 + 1.7 = 1.47$$

V = 13.916 + (-3.783) = 10.133

W = 19.75 + (-3.8) = 15.95
Z = 13.667 and falls within FROM and TO times chosen.

e)

$$h = \cos\left[180^\circ \times \frac{(13.667-10.133)}{(15.95 - 10.133)}\right] \times \frac{(3.44-1.47)}{2} + \frac{(3.44+1.47)}{2} - 2.40$$

h = -0.27 metre

NOTE: This height is with respect to mean sea level and not chart datum. The mean water level (K - 2.40) has already been subtracted.

5 *HORIZONTAL CONTROL*

5.1 EDM TRAVERSING

5.1.1 Introduction

With the advent of Electronic Distance Measurement (EDM) capability in the late 1950s, the use of traversing as a means to provide survey control increased enormously. Traversing could be used economically to provide control densification and became a particularly useful method of establishing map control in the mountains. This method of surveying was used extensively at Geodetic Survey prior to the development of the Doppler satellite and the inertial positioning capabilities in the mid-1970s.

EDM traversing is still used by Geodetic Survey, though much less frequently.

5.1.2 Instruments and Equipment

A theodolite and EDM unit are the principal instruments used for traversing.

Theodolite

Normally, one arc-second resolution theodolites are required to achieve second-order survey control accuracy. Three different models are currently in use at the Geodetic Survey: the Wild T2, Wild T2A and the Kern DKM2A. The latter two offer the advantage of an automatic compensator for indexing the vertical circle.

EDM Equipment

There are two groups of EDM instruments used for traversing: microwave and electro-optical. The basic design and operating principles of both types are explained in section 5.1.4. Microwave instruments in current use in the Geodetic Survey are the Tellurometer CA 1000 and Microfix 100C. The tellurometer MRA3 and earlier models were extensively used in the past.

a) Meteorological Instruments

Atmospheric pressure and humidity observations are necessary for computing "met corrections" to distances measured electro-magnetically. Three types of psychrometers are used to measure humidity - the sling, the electric and the digital.

The sling psychrometer consists of dry and wet bulb thermometers mounted to pivot about a handle. These may be swung through the air by hand at a constant rate to allow an even flow of air over the wet bulb. The second type has an electric fan which circulates air over both the wet and dry bulb thermometers. In both cases readings are taken after 2 or 3 minutes of operation so that a minimum wet bulb reading will be obtained. Electronic digital equipment provides a digital readout in less time.

Atmospheric pressure is usually measured using a light-weight aneroid barometer.

b) Targets - Sighting

Several types of sighting targets are commonly used for horizontal and vertical angle measurements. One type is a 12 V sealed beam light enclosed in a wooden box. The light is equipped with a rheostat to allow variation of brightness, a circuit breaker to permit flashing, and a pointing sight. There are two mounting assemblies, one to mount the light on top of a survey tripod for centering over a station for horizontal angle measuring; the second has a ball-joint and steel clamp for attaching to a tripod leg when simultaneous vertical angles are to be measured.

A second target type consists of a 12 V sealed beam in an aluminum frame. The frame has a flat aluminum lip which slides between the base of the transit and top of the tripod. When the transit securing screw is tightened the lamp assembly is also secure. This target is also equipped with a rheostat, a circuit breaker and a pointing sight.

A third target, also a sealed beam unit in an aluminum housing, is mounted on an inexpensive disposable tripod, usually left on the backsight station.

These targets are used for medium and long distance sighting (in excess of 1 km).

c) Tapes - Measuring

A three-metre and fifty-metre steel tape form part of the survey equipment. The former is for measuring instrument and target heights, the latter for measuring to nearby survey points, reference points, eccentric stations, etc.

d) Power Supply

Twelve-volt batteries supply power to electronic distance-measuring (EDM) equipment and target lights. The following types of batteries are used:

- disposable dry cells;

- nickel-cadmium (NiCad) batteries. These are reliable, clean, spill-proof and non-corrosive but they are heavy and relatively expensive;

- motor-cycle type lead-acid batteries. These have slightly less capacity, about half the weight of the NiCad and are about one-tenth the cost. This type is prone to spillage and is not suitable for aircraft transporting;

- rechargeable internal batteries. Most modern EDM instruments are equipped with these. They are usually adequate for short-term operation.

e) Traversing Equipment

Each member of an EDM traverse party needs the following equipment:

- 1 theodolite, usually Kern DKM2A, Wild T2 or Wild T2A

- 1 Kern or Wild Tripod

- 1 Kern or Wild Adapter

- 1 12 V sealed beam signal light

- 1 emergency signal mirror

- 1 CA 1000 Master Tellurometer

  or

- 1 CA 1000 Remote Tellurometer

  or

- 1 Microfix 100C

  or

- 1MRA3 Tellurometer

- 1 12 V battery
- 1 backpack
- 1 aneroid barometer
- 1 psychrometer
- 1 die set to stamp station numbers
- Supply of station markers
- Survival equipment for hinterland areas
- Target material for photo-identification
- 1 35 mm camera to photograph station sites on ground
- 1 radio receiver-transmitter such as a Motorola PT300 or SBX11
- 1 set of emergency flares
- 1 field notebook
- 1 first-aid kit, pocket type
- 1 compass

In addition, the survey party has a portable core drill and two photo-identification cameras.

Geodetic Survey usually operates three-person traverse crews eliminating the need to have unmanned backsight targets.

5.1.3 Traversing Elements

The following general elements should be considered in the initial planning and design of a traverse.

5.1.3.1 Accuracy

See Table 5-3, EDM Instrument Accuracies.

A survey station position is classified according to whether the semi-major axis of the 95 percent confidence region for position difference, with respect to adjacent stations of the traverse, is less than or equal to:

$$r = C\ (d + 0.2) \qquad \text{(S \& M, 1978)} \qquad (5\text{-}1)$$

where

r is the radius of the semi-major axis of the 95% confidence region in centimetres

d is the distance in kilometres to the adjacent station

C is a constant

For second-order horizontal control, C is assigned a value of 5, for third-order C = 12.

5.1.3.2 Traverse Design and Geometry

The size of the computed confidence regions for position differences depends on the accuracy of field measurements and the geometric configuration of the traverse network. The network must be homogeneous, have redundancy, and be well-shaped. The following factors should be considered when designing a network:

a) loop traverses (starting and terminating at the same point) should be avoided as they do not provide checks on possible scale errors within the traverses;

b) traverses must follow a relatively straight line in order to minimize the influence of the errors in angular measurement. Angular deflections should be less than 30° from the straight line;

c) short traverse courses should be avoided as it is difficult to carry an accurate azimuth through them. As a general rule, the ratio of the longest course to the shortest should be less than 5;

d) direct connections should be made to adjacent existing horizontal control stations. They serve to strengthen the network, to provide a check on azimuth, and to integrate the survey;

e) the absolute orientation of the traverse must be closely controlled. Starting and ending azimuths may be derived from direct connections to a previously established higher-order control network. In addition, control azimuths must be provided at least every six course along the traverse by independent astro or gyro observations, or by tying to an adjacent network.

5.1.3.3 Pre-analysis

Although the preceding guidelines increase the likelihood that second-order relative accuracy will be achieved, the final accuracy will depend to a large extent on the type of instruments and procedures used. An assessment of the accuracy that may be obtained on a proposed traverse can be derived from the results of a rigorous least square adjustment of the fictitious or simulated observations.
Model observations (distance and direction measurements) may be coded for a proposed traverse design and, together with a priori estimates of observation accuracy, may form the input to a simulated least squares adjustment. The computed 95% confidence regions from the simulation will give an indication of the relative accuracies likely to be achieved.

5.1.3.4 Reconnaissance

Pre-selection of traverse routes and station sites by ground reconnaissance is usually uneconomical. If the survey is to provide control for mapping, the mapping requirements largely govern route and station site selection.

Careful stereoscopic study of aerial photography will help to determine if traverse stations are intervisible and are accessible by car or helicopter.

5.1.3.5 Site Selection

Ease of recovery of a survey marker and ease of accurate photo-identification are important factors in site selection.

Markers should be placed at the highest point of land or where there is a distinctive identifiable topographical feature. Side-hills or extensive featureless areas are unsuitable for mapping control.

5.1.3.6 Station Marking

The type of survey marker used depends largely on ground conditions. A Geodetic Survey bronze tablet, cemented into a drill hole in bedrock or into a large boulder, is preferred.

A light-weight, portable, gasoline-powered, diamond bit drill is recommended for drilling in rock. Bronze markers are cemented into the drill-holes using quick-drying cement. If no rock is present, a one-metre minimum length iron bar with a stamped aluminum cap may be driven into the ground to serve as the marker. Other acceptable markers recommended by Geodetic Survey are listed in (S & M, 1978) and some others are listed in Appendix B, pages B-1 and B-2.

5.1.4 Electronic Distance Measurement

The term electronic distance measurement (EDM) includes both microwave and electro-optical types of measurements.

5.1.4.1 Principles

Microwave instruments use carrier wavelengths in the 1 cm to 10 cm range. This type of instrument is not significantly affected by a variety of atmospheric conditions - fog, haze, smoke, etc. Consequently, measurements may be made under adverse weather conditions.

Electro-optical instruments employ helium-neon laser light (visible), or infrared light (which is invisible and is produced by a gallium-arsenide diode) as carriers. Generally, shorter carrier wavelengths permit more accurate measurement.

EDM instruments utilize a modulated carrier frequency. This modulated carrier wave is transmitted from one end of the line to be measured, reflected at the other end back to the transmitting instrument where the phase difference between the transmitted and returned modulating frequency is measured.

The distance D between the transmitting instrument and the reflector is derived by the formula:

$$D = \frac{cT}{2n} \quad \text{(Saastamoinen, 1967)} \qquad (5\text{-}2)$$

where

D = distance in metres

c = velocity of light in a vacuum = 299,792,458 metres per second

T = observed time in seconds

n = refractive index along ray path

5.1.4.2 Measuring Procedures

Distance measurements by EDM should be made from each end of the line. Coarse distance readings and meteorological observations of pressure and wet and dry temperatures are taken at the beginning and end of distance measuring.

The following tabulation indicates the observing requirements for distance measurement using different instruments, and the accuracies to be expected. The accuracies are expressed as $\pm$(A+B), where A is a constant error and B is distance dependent, and reflect a one sigma estimate. The values of A and B have been derived from analysis of observed data obtained from field work with the instruments over several years and from the manufacturer's information. A double measurement is one measured from both ends of a line.

INSTRUMENT	ACCURACY ±A (cm)	 ±8 (ppm)	REMARKS	MAXIMUM RANGE KM
MRA 3	5.2	5.2	Double measurements 13 cavities	50-70
CA 1000	3.0	10.0	Double measurements 7 frequencies	30
Microfix 100C	1.5	3.0	Double measurements 10 frequencies	60
Ranger IV	1.0	2.4	20 measurements	12
Ranger V	1.0	2.4	20 measurements	24
Rangemaster II	1.0	1.2	20 measurements	60
Rangemaster III	1.0	1.2	20 measurements	60

Table 5-1 - EDM Instrument Accuracies

5.1.4.3 Measurement Corrections

The following corrections are applied to measured distances.

a) Refraction

The velocity of electro-magnetic wave propagation in a vacuum is 299,792,458 metres/second (Rapp, 1983). In an atmosphere the velocity is slightly less than this value. The ratio of the two velocities is the refractive index (n) of the medium through which the waves are travelling.

$$n = \frac{\text{velocity in a vacuum}}{\text{velocity in medium}}$$

$$n = 1 + \left[\frac{103.49\ (p-e)}{(t+273.15)} + \frac{86.26}{(t+273.15)}\left\{1+\frac{5748}{(t+273.15)}\right\}e\right]\cdot 10^{-6} \qquad (5\text{-}3)$$

where

p = atmospheric pressure in mm Hg

t = temperature in °C

e = partial water vapour pressure in mm Hg

Under normal conditions, this formula is accurate to ± 1 part in 10^7, and in extreme conditions it is good to ± 1 ppm.

The value of n for normal atmospheric conditions is approximately 1.000325 and is calculated from temperature, pressure and relative humidity readings taken at both ends of the measured line. A mean value for the refractive index is assumed to apply to the complete length of the line. Pressure to ± 3 mb and temperature to ± 0.2°C are measured . Programs for calculating atmospheric corrections for both microwave and electro-optical measurements are available for use with field computers.

b) Zero Error Correction

A zero error is the difference between the electronic centre of the instrument and the centering marks used to plumb the instrument over the survey station.

The internal distance the signal travels in the instrument is normally longer than the direct distance between the point of arrival of the signal and the centering mark. Manufacturers supply the zero error information that should be algebraically added to the measured distance to compensate for the difference. However, it has been found that the zero error may vary over the life of an instrument. This variance is normally small in electro-optical equipment but larger in the microwave types. It may be verified by measuring several distances on a calibration baseline. Distances varying from 50 to 1000 metres are recommended for electro-optical and 200 to 1000 metres for microwave type equipment.

If known distances are not available, the zero correction can be determined using a method of subdivided distances. Establish and mark four stations A,B,C and D, in a straight line on flat ground, see Figure 5-1.

The total distance AD and the distance AB, BC, CD, are measured with the instrument to be calibrated. Assuming that the zero correction (Z) is constant for each measured distance, the value can be calculated as follows:

$$2Z = AD - (AB + BC + CD) \quad (5\text{-}4)$$

If the distances are measured in all combinations, the zero error can be derived from:

$$2Z = AC - (AB + BC)$$
$$2Z = BD - (BC + CD)$$

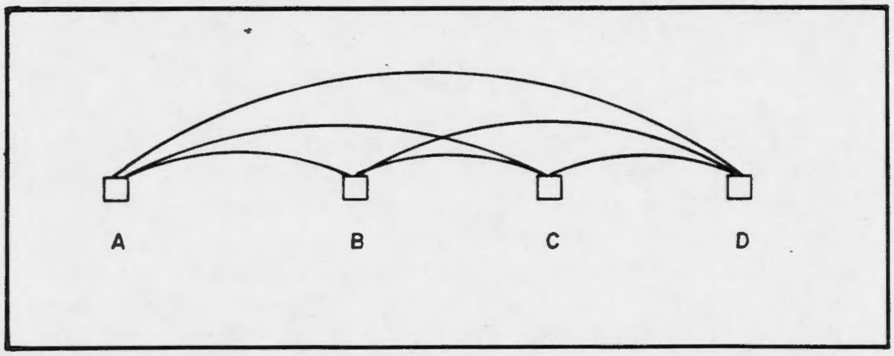


Figure 5-1 - Zero Error Determination

NOTE: The measured distances must be corrected for refraction and slope before calculating Z. The zero error is the combined error for a pair of microwave instruments and includes the reflector constant in the case of electro-optical instruments.

c) Frequency Calibration

Frequencies of EDM instruments are calibrated in the Geodetic Survey's electronics laboratory before and after every field season and whenever an instrument is returned to the lab for servicing. Frequency corrections are applied at the end of the field season.

d) Multiple Path Errors (Ground Swing)

Error due to multiple path may occur when measurements are being made over highly reflective surfaces such as ice, water, etc. The reflected rays do not follow the most direct path between the transmitting and receiving units. A series of readings observed over a range of carrier frequencies tends to vary cyclically about the true value. The multiple path effect is more pronounced with a wider signal beam. With an MRA-3, for example,errors up to 50 cm may result from multiple path. When strong multi-path signals are detected their effect may be reduced by moving one or both EDM units horizontally or vertically from the path of secondary reflections.

e) Eccentric Error

EDM equipment should be levelled for all measurements. If the instrument is not level, an eccentric error will be introduced.

f) Reflector Constant

The reflector constant is the effective distance between the physical and optical center of the reflector. Because of transmission characteristics of the various reflector materials this may or may not be the actual physical difference. Various reflector types are listed below with their associated constants.

Reflector Type	Reflector Constant
AGA Corner Prism	-0.03 m
Plastic Reflector	variable
White Bond Paper	0.00m

Table 5-2 - Reflector Constants

A reflector constant may be determined by using the reflector to measure a distance which is already known accurately.

Reflector Constant = Distance (known) - Distance Measured corrected for meteorological effects, zero error, etc.) (5-5)

Another method which can be used when no known distance is available:

1) place a sheet of white bond paper at a convenient distance (50 metres) from the instrument and take a set of observations;

2) place the reflector to be calibrated in precisely the same position and take a set of observations;

Reflector Constant = Distance (paper) - Distance (reflector)

5.1.5 Horizontal Angle Measurement

NOTE: For vertical angle measurement, refer to section 4.2, Trigonometric Heighting.

5.1.5.1 Introduction

The direction method for measuring horizontal angles is recommended. In this method the direction to each station is measured clockwise from one line which is considered the initial line. The angle between any two points is the difference between the directions to them.

Terminology

Observation - the horizontal circle reading for one pointing of the theodolite.

Set - a series of observations on two or more targets, so that:

a) the setting of the horizontal circle remains constant,

b) an observation is made on each target successively in clockwise order beginning with the initial line. With the telescope reversed (on reverse clamp) each target is observed consecutively in counter clockwise order beginning with the target last sighted.

5.1.5.2 Angle Measuring Technique

Second-order horizontal angle measurement specifications require that at least six sets of angles be measured at each station (S & M, 1978). The practice at Geodetic Survey is to measure eight sets. These are evenly distributed on the horizontal circle to minimize circle graduation errors.

To eliminate the small error caused by creep in some instruments, the following procedure has been adopted.

Three stations, A, B and C are occupied with the theodolite set up at B. The angle ABC is measured (four sets) using BA as the initial line. Then the complementary angle CBA is measured (four sets) using BC as the initial line (Figure 5-2).

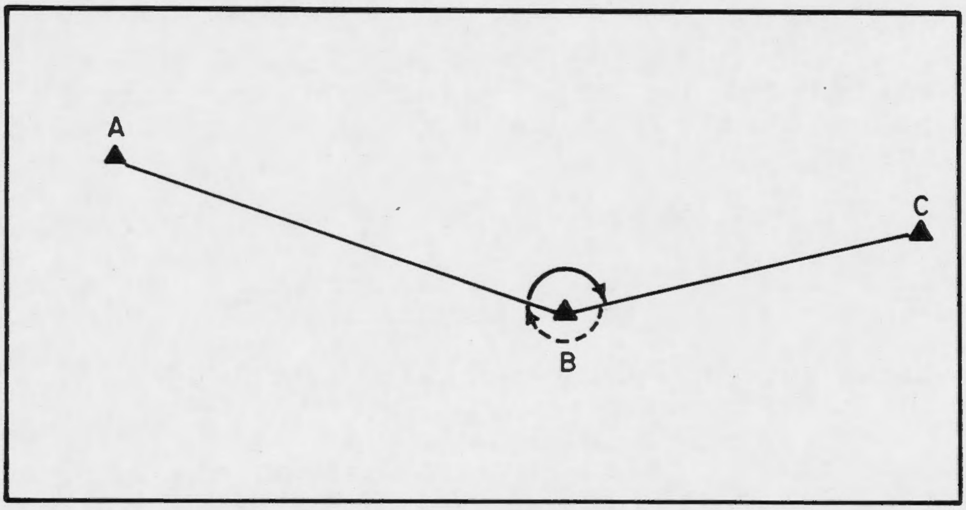


Figure 5-2 - Angle Measuring Technique to Determine Circle Creep

By using this technique, any error caused by horizontal circle creep may be estimated by differencing the sum of the two observed angles from 360°. This refinement improves loop closures for only a slight cost in extra time. If the eight sets of angles have a spread greater than 10 seconds, four additional sets are observed; two each on the different backsights.

5.1.5.3 <u>Observing Procedures</u>

1) Identify the station marker and record accurately the information stamped on the marker. Positive identification is essential. A photo or tracing of the marker showing the markings removes all doubt.

2) Set up the tripod and instrument over the marker, centre and level the instrument. Check for stability by pointing on a target walking around the tripod and verifying that the cross-hair has not moved off the target.

3) Point on the initial station and set the circle to the appropriate setting. The micrometer does not have to be set on a prescribed setting but should vary from set to set.

4) Repoint the telescope by turning the slow-motion screw clockwise against the pressure of the spring. Fine pointing to all targets must always be done in this manner.

5) Read the circle and micrometer and record the readings.

6) Point on the next station to the right. Read the circle and micrometer and record the readings.

7) Continue this procedure clockwise until the last station in the group has been read.

8) Transit the telescope and repoint on the last station sighted. Read the circle and micrometer and record the readings.

9) Sight and read on each station, moving counter-clockwise until the initial station is reached.

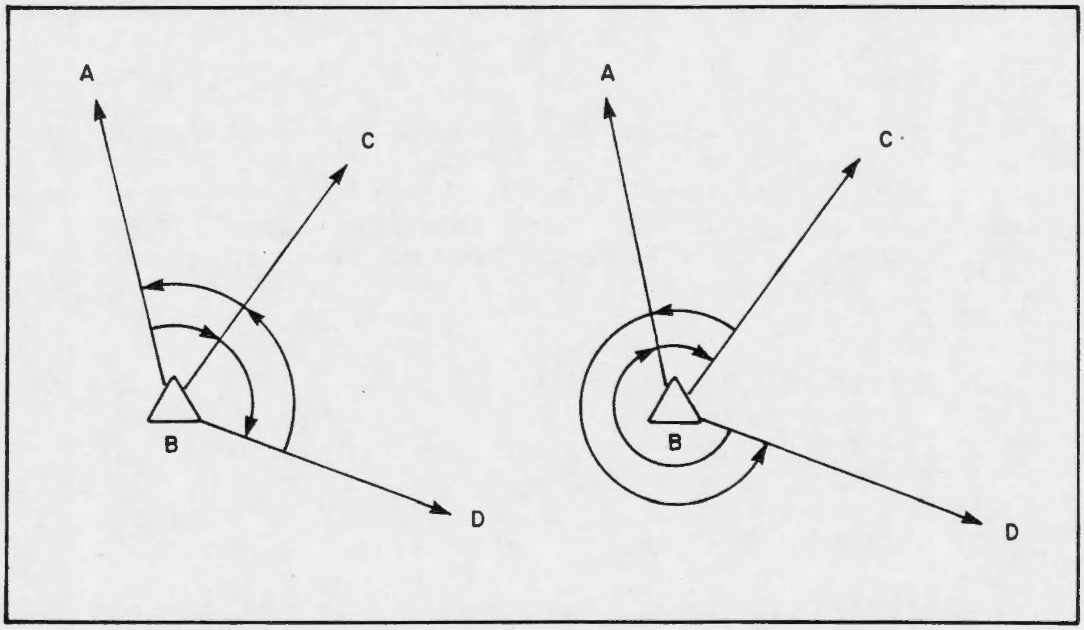


Figure 5-3 - Horizontal Angle Measurements

(1) first four sets: A = initial station
D = final station

(2) second four sets: D = initial station
C = final station

This completed the first set. Advance the circle setting by 22° 30′ (8 sets are to be observed) and start the second set. Advance the same amount for every consecutive set until 4 sets are measured. After half the measurement is completed (4 sets), point on the final station, set the circle to the next setting, and measure 4 sets of complementary angles clockwise.

5.1.5.4 Common Sources of Error

Assuming that the theodolite is in good adjustment, the following procedures are designed to eliminate or minimize errors.

SOURCES OF ERROR	CORRECTION PROCEDURE
a) Centering	Centering rod bubble or optical plummet should be checked frequently and the instrument recentered if necessary. This must only be done between sets.
b) Line of collimation not coincident with telescope axis	A set consisting of direct and reverse (clamp left andclamp right) readings is measured.
c) Parallax	Careful focussing.
d) Circle graduation error	Minimized by observing a series of sets (6 to 8) spread evenly around the horizontal circle.

Table 5-3 - Horizontal Angle Error Sources

5.1.6 Field Data Recording

A loose-leaf field record notebook is used to record field survey data. Individual note-keeping formats are used for recording horizontal angles, vertical angles, distance measurements, spirit levelling, photo-identification details, etc. and are prepared in bilingual form. Each page consists of an original and a copy as a precaution against loss.

It is important that all headings of the field notes be completed including - occupied station number, target station number, date, observer's name, instrument numbers, instrument height, target height, eccentricities, etc. A sketch should show the relation of the station occupied to connecting stations, subsidiary station (side shots), reference marks, topographic features, etc.

Forms used for EDM traversing are:

Horizontal Angles	- Form SCS 78-2 (Appendix D, p.D2)
Vertical Angles	- Form SCS 78-5 (Appendix D, p.D4)
Tellurometer Distance Record	- Form SCS 78-3 (Appendix D, p.D3)

5.1.6.1 Field Data Security

Survey notes are checked at the end of each days's work and initialled by the checker. An abstract of the data is prepared and the original copy of the notes is sent to HQ.

The duplicate copy is retained in the field-office and is used for field computations. This procedure is followed to provide backup notes in case of accidental loss or destruction of field data.

5.1.7 Personnel

EDM traverse parties consist of either two or three persons and may be serviced adequately by one helicopter. Flying time is greater for the three-person crew. If a flying contract is based on minimum flying hours per month, both types of crews can operate well within the minimums. Generally, the three-person crew will be more productive.

5.1.8 Field Computations

Field reductions and computations are required to determine preliminary positions and to detect errors in observed data. A small programmable computer is used for most field computations.

5.1.8.1 Distance Measurement Reduction

Section 5.1.4.3 explained the computations for zero and refraction corrections for both microwave and electro-optical measuring equipment. These corrections must be applied before measured distances are reduced further. EDM measurements must be corrected for slope, then reduced to sea level distances before they may be used to compute positions on the reference ellipsoid. In field computations, the geoid/ellipsoid separation is usually ignored.

The general reduction model used is:

$$S + D - [\frac{DH}{R}] - [\frac{h2}{2D} + \frac{h4}{8D3}] + A.C. \text{ (Saastamoinen, 1967} \qquad (5\text{-}6)$$

where

S = sea level

D = distance, corrected for meteorological observations

R = radius of curvature = $6356584/(1.0 - 0.0067866 \sin^2\phi)$

H = mean elevation $\frac{(h_1 + h_2)}{2}$

h = elevation difference $(h_1 - h_2)$

h_1 = elevation station 1 + height of instrument 1

h_2 = elevation station 2 + height of instrument 2

ϕ = latitude of working area (degrees only)

(Saastomoinen, 1967)

$$A.C. = \text{arc curvature correction} = (1\ \kappa)^2 \frac{K^3}{24R^2} \qquad (5\text{-}7)$$

(Saastomoinen, 1967)

where

κ = the coefficient of refraction

K = the sea level chord distance $\approx D$

For microwave, an average value of $\kappa = 0.25$ may be used, then

$$A.C. = \frac{K^3}{43R^2} \approx 5.7\ D^3 \times 10^{-16}$$

$$A.C. = \frac{K^3}{43R^2} \approx 5.7\ D^3 \times 10^{-16}$$

For light rays, an average value of $\kappa = 0.2$ may be used, then

$$A.C. = \frac{K^3}{38R^2} \approx 6.5\ D^3 \times 10^{-16}$$

In equation (5-6), the expression

$$\left[\frac{DH}{R}\right]$$

reduces the slope length of the line from the mean elevation of the line, H, to the slope length at sea level.

$$\left[\frac{h^2}{2D} + \frac{h^4}{8D^3}\right] \qquad (5\text{-}8)$$

reduces the sea level slope length to the horizontal chord distance.

The last term of the model, arc curvature, is added to sea level horizontal chord distance to determine the sea level arc distance. Arc curvature correction differs for microwave and electro-optical equipment.

5.1.8.2 Horizontal Angle Reduction

No corrections are applied to horizontal angles for field computations. A standard deviation of 2.0 seconds is satisfactory from six sets of observed angles. (Note: It is common practice at Geodetic Suvey to observe eight sets). The reduction of vertical angles is explained in 4.2.5.

5.1.8.3 Eccentric Measurement Reduction

Corrections to distances and angles measured eccentrically are based on approximate formulae. They are only accurate for relatively short offsets on long lines, where the angle at the distant station between the main station and eccentric is less than 30 minutes of arc. Wherever possible, eccentric observations should be avoided.

a) Reduction of distance measured eccentrically

The following approximate formula may be used to reduce distances measured eccentrically:

$$S = S \ + d \cos A \qquad (5\text{-}9)$$

where

S = required distance, station 1 to station 2

S = observed eccentric distance, station 1 to station 2

d = eccentric horizontal distance, station 1 to eccentric station

A = angle measured at main station from station 2 to eccentric station

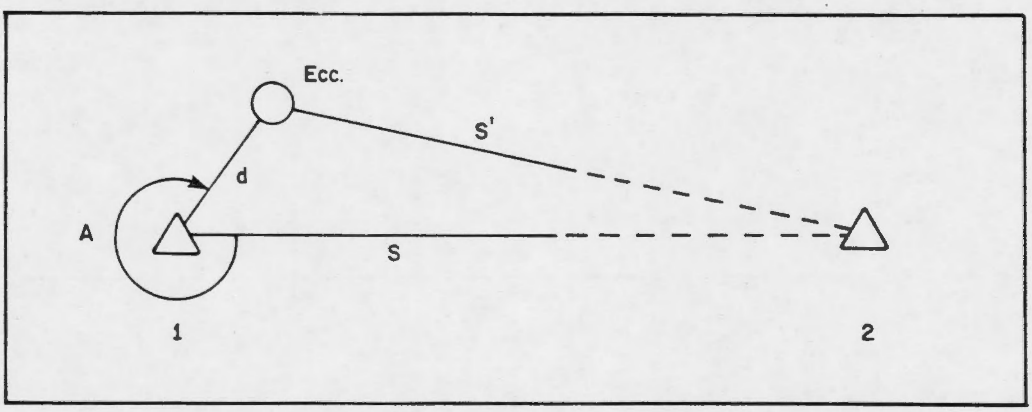


Figure 5-4 - Reduction of Eccentric Distances

b) <u>Reduction of directions measured eccentrically</u>

This becomes necessary when the theodolite is placed somewhere other than on the main station. The correction is caluclated by the following formula:

$$C = \frac{d \sin A}{0.0048481 S_{12}} \text{ arc-seconds} \tag{5-10}$$

where

C = correction in seconds to the direction measured from the eccentric station = ξ

d = the eccentric distance (horizontal) in metres

A = angle measured to station 2 from the eccentric station and referred to the main station (1) as initial

S_{12} = distance of line between station 1 and target station

NOTE: This formula is accurate to ±0.01 seconds for corrections smaller than 30 minutes of arc.

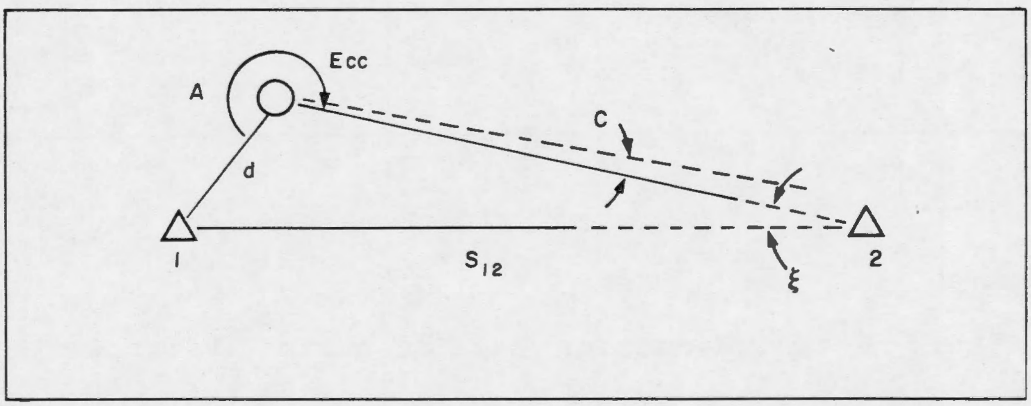


Figure 5-5 - Reduction of Eccentric Directions

c) Reduction of directions measured to eccentric targets (Fig. 5-6)

Direction corrections to eccentric targets are computed from the formula:

$$C = \frac{d \sin A}{0.0048481 S_{12}} \text{ arc seconds} \tag{5-11}$$

where

A = angle measured at the target station from station 1 as initial to the eccentric target

C = correction in seconds to the direction measured to the eccentric target from station 1

d = horizontal distance from target station to eccentric target

S_{12} = horizontal distance between main station and target station

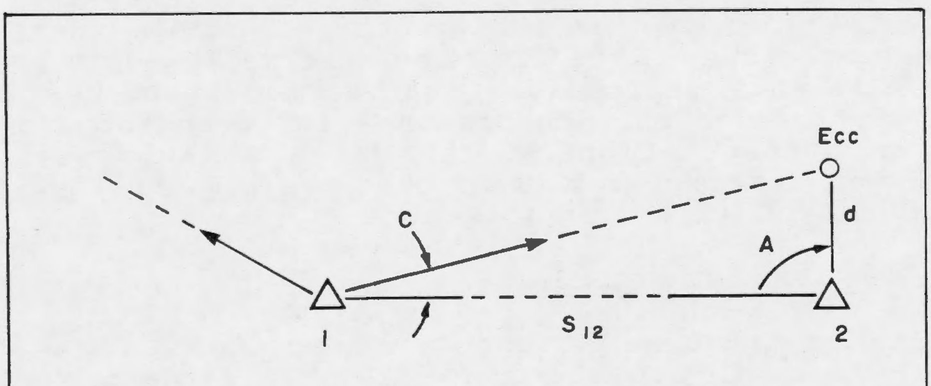


Figure 5-6 - Reduction of Direction to Eccentric Target

5.1.8.4 Miscellaneous Computations

Two other computations related to EDM traversing are required in the field. These are:

a) the Direct geodetic computation where, given the latitude and longitude of a station and the azimuth and reduced sea level length to a second station, the coordinates of this second station and the back azimuth are obtained.

b) the Inverse geodetic computation where, given the latitude and longitude of two points, the distance and azimuth between them is obtained.

The reference ellipsoid used for the computatins has been the Clarke 1866, and will become the WGS 84 after 1986. Geodetic Survey has developed programs for use with portable TI-59 computers for these computations by field-survey parties.

5.1.8.5 Traverse Misclosures

As stated in 5.1.3.1, for second-order traversing the required standard of relative positional accuracy is an error ellipse whose semi-major axis, expressed at the 95 percent confidence level with respect to other stations in the traverse, is less than 5 (d + 0.2) cm, where d is the distance in kilometres between stations (S & M, 1978).

Two empirical "rules of thumb" may be used as quality checks. The linear misclosure must not be greater than 1:40 000, and the azimuth misclosure not greater than $5''\sqrt{N}$, where N is the number of sides in the traverse. Normally, both are readily obtained when recommended observation practices are used. The misclosures must be obtained for both forward and back traverse computations. The linear misclosures will be different but the azimuth misclosures will be the same if no computation blunders exist.

5.1.8.6 Checking Procedures

A blunder in observations can sometimes escape detection (e.g. an angle observed to a wrong target). The field reduction of data should not be considered complete until the reduced directions and distances have been scrutinized for consistency. The main checking procedure involves the calculation of azimuth and position misclosures along the traverse.

The sum, in degrees, of the inner angles in a closed figure is equal to (N -2) x 180° + spherical excess, where N is the number of angles. If the azimuth misclosure is greater than 5"√N, then either a blunder or systematic error accumulation may be suspected.

Assume a traverse begins at a known point (A) and proceeds to another known point (G), starting and finishing at known azimuths, and a blunder is present in the angle at station D (see Figure 5-7).

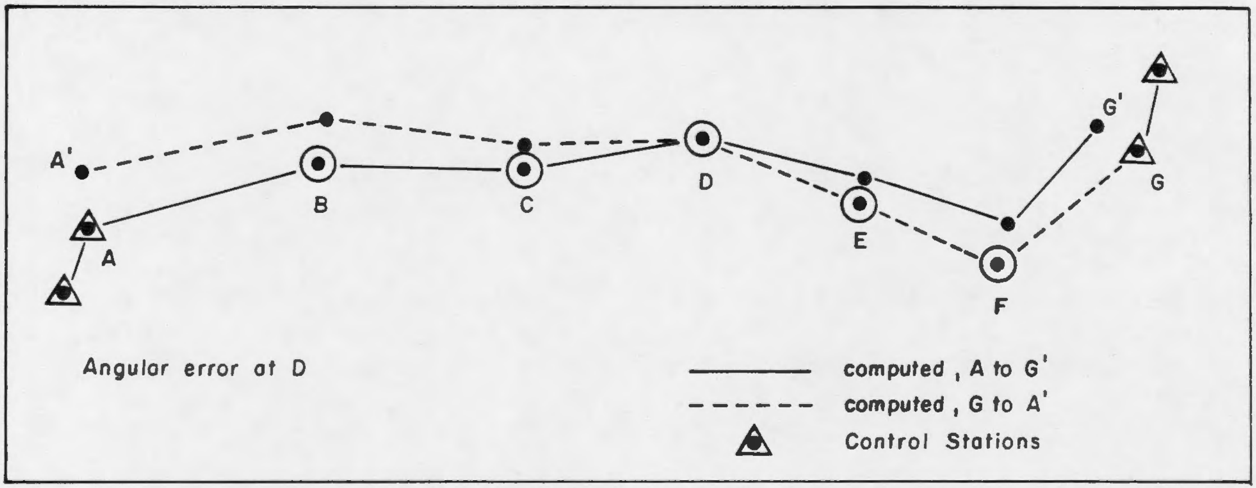


Figure 5-7 - Isolation of an Angle Blunder

The coordinates of G are calculated progressively starting from the position A (assumed known) and the starting azimuth (also assumed known) and terminating at a computed point G, i.e. G'. Theoretically, the differences between the coordinates of G and those computed should equal zero. The actual misclosures ($\phi_G - \phi_{G'}$) and ($\lambda_G - \lambda_{G'}$), expressed in metres, should be better than 1:40 000. This ratio is computed using the formula:

$$\frac{[(\phi G' - \phi G)^2 + (\lambda G' - \lambda G)^2]^{1/2}}{L} \qquad (5\text{-}12)$$

where L = the total length of the traverse from point A to G,

$(\phi G' - \phi G)$ = latitude difference converted to metres

$(\lambda G' - \lambda G)$ = longitude difference converted to metres

To compute traverse misclosures, observed horizontal angles must be used. The angles must never be adjusted prior to evaluation of the accuracy of the traverse.

Angular blunders in a traverse may be detected by computing the traverse coordinates from both ends. A blunder may have occurred where the coordinates of a station have the same values, see Figure 5-7.

5.1.9 Least Squares Adjustment

After preliminary computations are checked, the observed data are used as input to the horizontal least squares adjustment program "GANET".

Input consists of three sets of information: geographic positions, observed directions/azimuth and sea level reduced distances with associated weights or variances.

a) Geographic Coordinates

Station numbers and names with published, calculated, or map scaled posisitons for each station involved in the survey are input. Code 04 input format is used.

b) Observed Directions/Azimuths

Observed directions and azimuths in clockwise order are input for each station. The code for azimuths is 03, directions are coded 01.

c) Sea Level Reduced Distances

Sea level distances between stations are coded 02.

Additional information is available from the guide for program "GANET" (Beattie, 1978).

5.2 Doppler Satellite Positioning

5.2.1 Introduction

The navy Navigational Satellite System (NNSS), or Transit System, was developed from research initially carried out by F.T. McLure in the United States. A process was developed whereby ground positions could be determined, using the Doppler effect on radio signals from satellites in a known orbit.

At present (1986), there are six operational Transit satellites. They travel about 1000 km above the earth's surface in a polar orbit, and complete an orbit in about 1 hour 46 minutes. During the period of one orbit, the earth rotates about 26.8° (Boal, 1983).

To an observer on the earth at 45° latitude, an example of the apparent motion of successive passes of satellite 50 follows:

PASS #	ELEVATION AT CLOSEST APPROACH (C.A.)	RISE TIME	CLOSEST APPROACH (C.A.) TIME	SET TIME	DIRECTION
1	2°	5:41	5:45	5:48	E 146°
2	31°	7:27	7:36	7:45	E 170°
3	52°	9:15	9:25	9:35	E 170°
4	09°	11:05	11:12	11:19	W 211°

Table 5-4 - Apparent Motion of Successive Satellite Passes

The first pass, for example, is travelling from north to south on the east side of the observer's meridian, it rises above the horizon at 05:41 and reaches a maximum elevation angle of 2° at closest approach to the observer at 05:45 and drops below the horizon at 5:48.

Only those passes for which a satellite rises above 15° elevation at closest approach are used for positioning. In the example, there are two potenitally usable passes. As the earth rotates 180° a similar situation exists except that, for this satellite, the direction is now from south to north.

Because of convergence of the satellite orbits over the north and south poles, the number of good passes increases as the observer's latitude increases. The number of potenitally good passes per satellite per day vAries from about $2\frac{1}{2}$ at the equator to more than 13 at latitudes 74° to 90°.

The Transit System is operated by the U.S. Navy Astronautics Group at Point Mugu, California. Additional tracking stations are in Maine, Minnesota and Hawaii. New predicted orbital data from these tracking stations are injected into the satellite memory at 12 to 16-hour intervals. These satellite orbital parameters are then transmitted back to earth every 2 minutes by the satellite until new data is injected.

The NNSS was first used by Geodetic Survey in 1973, and since that time has been used to provide first-order horizontal control and secondary survey densification.

5.2.2 Principles

The concept and technology of Doppler satellite positioning is complex and beyond the scope of this manual. Readers who require detailed information are referred to (Boal, 1983). A brief explanation follows.

The satellites are in known orbits, that is, the position of a satellite is known at any epoch. A "Doppler count" is accumulated in the receiver for each 4.6 second interval of time. A new accumulation is started every 2 minutes. This count is the difference, on each of the frequencies, between the number of cycles of the radio wave received from the satellite and the number generated by the receiver's reference oscillator in a 4.6 second period. These counts, along with orbital data and timing information from the satellite, are recorded on cassettes and later processed by sophisticated computer programs. Since the Doppler count is proportional to the change in range to the satellite, distance differences may be derived between the satellite and the receiving antenna every two minutes. To increase the number of "measurements", most software makes use of 30-second rather than two-minute accumulated Doppler counts. Thus, from a known orbit and computed distance differences, the coordinates of a receiving antenna can be determined. To resolve ambiguities, the surveyor needs to know whether his position is east or west of the orbit plane.

The observation techniques consist of point positioning and translocation using short arc or semi-short arc reduction procedures. Point positioning requires one receiver only. Translocation requires two or more receivers and occuptation of one or more base (known) stations at the same time the required stations are occupied.

All data collected by Geodetic Survey is processed in semi-short arc mode using program GEODOP (Kouba and Boal, 1975).

5.2.3 Instrumentation

Several makes of Doppler satellite receivers are manufactured in North America. At present, the Geodetic Survey uses those made by the Canadian Marconi Co. Ltd., models CMA 722B and CMA 761.

Data is collected on a digital cassette unit (DCU).

The CMA 722B receivers operate on an 115-volt A.C. power source and require frequent attention when in operation. The CMMA 761 receivers require a 12-volt power source and may be operated unattended under the receiver's microprocessor control.

The equipment at each "automatic" or unattended station includes an antenna, a CMA 761 receiver, a DCU and an environmental sensing unit (ESU) to collect meteorological information. the CMA 722B cannot be used with an ESU and "mets" must be observed conventionally.

Geodetic Survey's CMA 722B units have been interfaced with Apple II computers to automate them for use on base stations. They record both "raw" (4.6 second) and majority-voted (30-second) data (Quek et al, 1985).

5.2.4 Field Procedures

When translocation techniques are used, the selected base stations are normally part of the primary or secondary Doppler densification networks and are selected for a project on the basis of accessibility, power supply, accommodation for equipment and personnel, proximity to the survey area and the field headquarter, and freedom from electromagnetic or physical interference. Several methods of observing may be used. For details, the reader is referred to (Boal, 1983).

The precise crystal oscillators in the receiver must be completely warmed before usable satellite data can be collected. Experience has shown that this requires continuous power to the oscillator of the 722B for about 72 hours and to the 761 for about 24 hours.

A CMA 722B receiver searches continuously by sweeping a frequency range and will lock onto and collect data from any Transit satellite. Satellites that conflict with other Transit satellites may be included. The CMA 722B/Apple II receivers can be programmed to lock onto selected satellite passes.

The CMA 761 receiver software can predict the rise time and elevation angle of each satellite pass and selects those passes that will likely provide good data. The software will turn the receiver on, pick up and track a satellite, store raw data on cassette tape, majority-vote and store data in computer memory, attempt a 3-D position fix, and then turn the receiver off to wait for the next pass. The number of successful passes, good fixes and other statistics are indicated on a display that can be examined by the operator at any time.

At each new station at least 40 good fixes, as determined by the CMA 761 software, should be collected. This requires from 2 to $3\frac{1}{2}$ days at 45° latitude and from $1\frac{1}{4}$ to $2\frac{1}{2}$ days at 75° latitude. The two main factors affecting the number of good fixes per day are theconflicts between satellites and the amount of data rejected due to ionospheric activity.

Experience has indicated that the number of good fixes, as a percentage of the total number of passes identified for tracking over a three-day period, can vary from 55 to 85 percent. Also, it has been found that all receivers working in an area at the same time show about the same percentage on any given day.

The Geodetic Survey specifications required 20 passes at each new station which are in common with both base stations, in the final multi-station solution.

During post-processing, coordinates are computed for the location of the Doppler antenna. The antenna, mounted on a tripod or tower, must be plumbed over the survey marker, and the height of the antenna recorded so that the reduced coordinates are those of the survey marker. See Appendix D, p. D15 for sample field station note form (PHC-76, 1985) and explanation.

5.2.5 <u>Computations</u>

Field checking of the CMA 722B data may be accomplished by reading the data using an HP-2100 computer. The CMA 722B receiver also has a self-test feature which simulates a satellite pass to check all parts of the circuitry.

Data collected by the CMA 761 receivers are first automatically checked by the receiver software at the station, by attempting to compute a 3-D fix after each pass.

The cassettes are checked in the field-office to verify that they are readable and that raw data have been correctly recorded. This can be accomplished by reading the data into a CMA 761 receiver or a mini-computer for processing with a 3-D position fix program.

The HP-9816 computer is now used to compute single or multi-station position fixes as a check on the data from the CMA 761 and CMA 722B/Apple II receivers. The HP-9816 multi-station program (GEODOP) is limited to about 5 stations (GSD, 1985).

Before shipping the data to Ottawa, the cassettes from both types of receivers are duplicated in the field as insurance against loss. An alternative method of safeguarding the CMA 761 data is to "dump" the passes of majority-voted data from the receiver slave memory to a separate cassette. This cassette can then be retained in the field when the raw data cassette is shipped to Ottawa.

Post-processing of the observed Doppler satellite data involves four major steps:

a) <u>majority-voting</u> (MJV)
to condense and edit the raw data recorded by the receiver,

b) <u>prepocessing</u> (PREDOP) (Lawnikanis, 1976)
to screen out geometrically unacceptable passes, to include meteorological information and to prepare adjustment input data;

c) <u>single station solution</u> (GEODOP) (Kouba, Boal, 1975)
uses all passes recorded by a receiver at a particular site to screen out statistically unacceptable data, to evaluate receiver performance, and to prepare the edited data file;

d) <u>multistation solution</u> (GEODOP)
uses data files retained from step (c) to produce a three-dimensional network whose coordinates are consistent for all stations observed during a particular time period. These coordinate values have to undergo further transformation to bring them to the local geodetic coordinate system being used at the base stations.

5.2.6 <u>Accuracies</u>

At the 95% confidence level, using translocation methods, first-order results can be obtained between Doppler stations at spacings greater than about 58 km, and second-order at spacings greater than about 22 km. Prerequisites are suitable base stations, meteorological data for each station, and a minimum of 20 good passes in common between each new station and both base stations.

5.3 **Inertial Surveying**

5.3.1 Introduction

The Inertial Survey System (ISS), used by the Geodetic Survey, was manufactured by the Litton Corporation, California. The system was originally intended to provide rapid economical mapping control, with accuracies of two to three metres. Subsequent developments in adjustment software and survey procedures yield results of sub-metre accuracy.

In 1976, the first large grid-type project using ISS was undertaken in Southern Alberta. The data was simultaneously adjusted and classified as second or third-order using error ellipse information. Second-order relative accuracy is now achieved if procedural guidlines are strictly followed and station spacings are not less than 8 km.

5.3.2 Principles

The ISS uses accelerometers to measure acceleration along each of three orthogonal axes, gyroscopes to maintain the orientation of the system, and computer software to measure and double integrate the accelerometer information into distance travelled. Thus the ISS measures positional change from one station to the next. By starting and ending on points of known coordinates, the sytem can interpolate between them, and provide coordinates of the intermediate points.

A more detailed description of the operating principles is available from Geodetic Survey on request.

5.3.3 Instrumentation

The Litton Auto Survey System (LASS II), used by the Geodetic Survey since 1984, comprises five major components.

a) Inertial Measuring Unit (IMU)

This component contains an inertial platform consisting of two, two-degrees-of-freedom gyros and three orthgonally-mounted accelerometers set in four gimbal mounts, which isolate the platform from all horizontal and vertical motion. The IMU also contains power supplies, torque motors and other components necessary to maintain the inertial platform in constant attitude relative to its reference frame. The IMU senses vehicle motion relative to its reference frame, the

Data Processing Unit (DPU) assimilates and evaluates data from the IMU and sends information back to the IMU and to other components.

b) Data Processing Unit (DPU)

This component is a digital computer which contains 32K work memory, a control processor, input/output control logic and a power supply.

c) Control and Display Panel (CDU)

This unit is the interface between operator and system. It allows the operator to enter and recall data and to monitor the system status and pertinent functions. It also provides a visual display of the system's status.

d) Data Storage Unit (DSU)

The DSU is a mgentic cassette recorder. It has three main functions: record the survey data, record computer software programs and load the software programs into the computer.

e) Power Supply Unit (PSU)

The PSU converts +28 volts D.C. auxiliary input into 28 volt D.C. regulated, 28 volts D.C. unregulated, 115 volt A.C. 400 Hz, 26 volt A.C. 400 Hz, and 5 volts D.C. It supplies the appropriate power to all other system components.

5.3.4 Auxiliary Equipment

a) Mini-cassette Control Unit (MCCU)

Its primary function is to act as an interface permitting communication with the computer and the loading of diagnostic programs, calibration data and operational programs.

b) Alternators

These 28 volts D.C., 100 amp. alternators supply power to the ISS when coupled with a 24-28 volts battery. In developed areas, they are mounted on vehicles; in remote areas, on 7 1/2 HP or 10 HP Briggs & Stratton engines.

c) <u>Racks</u>

Racks, approved by Transport Canada, have been designed for ISS equipment installation in a Hughes 500 helicopter. They are easily adaptable to ground vehicles.

d) <u>Power Panel</u>

This was specially designed to permit use of the system in a helicopter. It consists of two diodes and cables to connect the system PSU and the aircraft 28 volts power. The main purpose of the diodes is to protect the aircraft generator and auxiliary generators when changing from one power source to the other.

e) <u>Offset Device</u>

This consists of a protractor mounted on the aircraft antenna. The center of the protractor is set up as the system reference point, and offsets (bearing, distance, elevation) to a survey point are measured from this reference point using a measuring tape and plumb bob.

A digitized Eccentricity Measurement System (EMS) is being developed at Geodetic Survey to improve accuracy and reduce the possibility of human error in recording. The unit consists of three shaft encoders for angles, a position displacement transducer for distance, a microprocessor, a printer and an inclinometer. The unit is mounted on the helicopter and a retractable flexible wire is pulled out and put on the station marker. When a switch button on the EMS is pushed, the slope distance, horizontal angle and vertical angle are automatically entered in the miroprocessor and the bearing angle, horizontal distance and difference in elevation are printed out. If the station marker is below ground level, a 0.5 metre rod is used for access to the marker and this is compensated for in the processor. The inclinometer automatically compensates for any mislevellment of the system (e.g. helicpter not level).

f) <u>DC Power Supply</u>

This is a 220 volts, 3-phase input and 28 volts D.C., 150-ampere output regulated power supply. This unit is used only in the lab for system maintenance, calibration and personnel training.

g) D.C. Power Supply (Christie)

This is a 115 volts input, 28 volts D.C., 50-ampere output. It is used in the field for maintenance and continuous running of the system during testing and static calibrations.

Other miscellaneous equipment includes:

- a metal detector to help find buried markers,
- measuring tapes, 3-metre and 100-metre,
- plumb bob.

A) Forward and Back Agreement:

Line Length	Delta H, ϕ, λ
< 80 kms.	1.50 m
80-100 kms.	1.75 m
> 100 kms.	2.00 m

B) On lines flown more than once (forward and back) the means should agree to:

< 80 kms.	1.00 m
80-100 kms.	1.20 m
> 100 kms.	1.50 m

C) Mean of cross checks and control checks:

All line lengths	≤ 1.00

Table 5-5 - ISS Field Specifications

5.3.5 Field Specifications

Geodetic Survey uses inertial surveying primarily to establish second and third-order multi-purpose survey control. To achieve this accuracy, procedures and specifications have been developed to overcome system limitations and to provide survey redundancy while still maintaining an acceptable cost/production balance.

a) Site Preparation

Before an inertial survey is begun, marker sites must be selected and prepared. For detailed specifications refer to "Specifications for ISS Site Preparations" (Geodetic Survey, 1980)

b) Straight Line Traverse

Because of the heading sensitivity of the Litton Inertial Survey System, all traverses must be run in a straight or nearly straight line to achieve maximum accuracies. The maximum deviation may be computed using the following empirical formula:

$$d = \frac{\ell}{12}$$

where

d = deviation in kilometres (perpendicular distance from station to line joining any two stations on the traverse)

ℓ = distance between any two stations on the traverse in kilometres.

c) Traverse Lengths

The two main components of the inertial survey system, gyros and accelerometers, have inherent errors which accumulate with time. These errors can be corrected or controlled over short periods by the software and by carefully following recommended survey procedures. The maximum recommended traverse length is 80 to 100 kilometres with at least one check point located on the traverse, when the system is mounted in a helicopter.

d) Zero Velocity Update (ZUPT)

A procedure to control error growth is used whereby the system is brought to a null or zero velocity state at regular intervals (i.e. 4 minutes). With helicopter transport this relates to a station spacing of about 10 km.

e) Station Spacing

The spacing of stations is not a limitation of the system and can vary with the user's requirement. The relative accuracy station to station may, however, be less than second-order with spacings of less than about 8 kilometres.

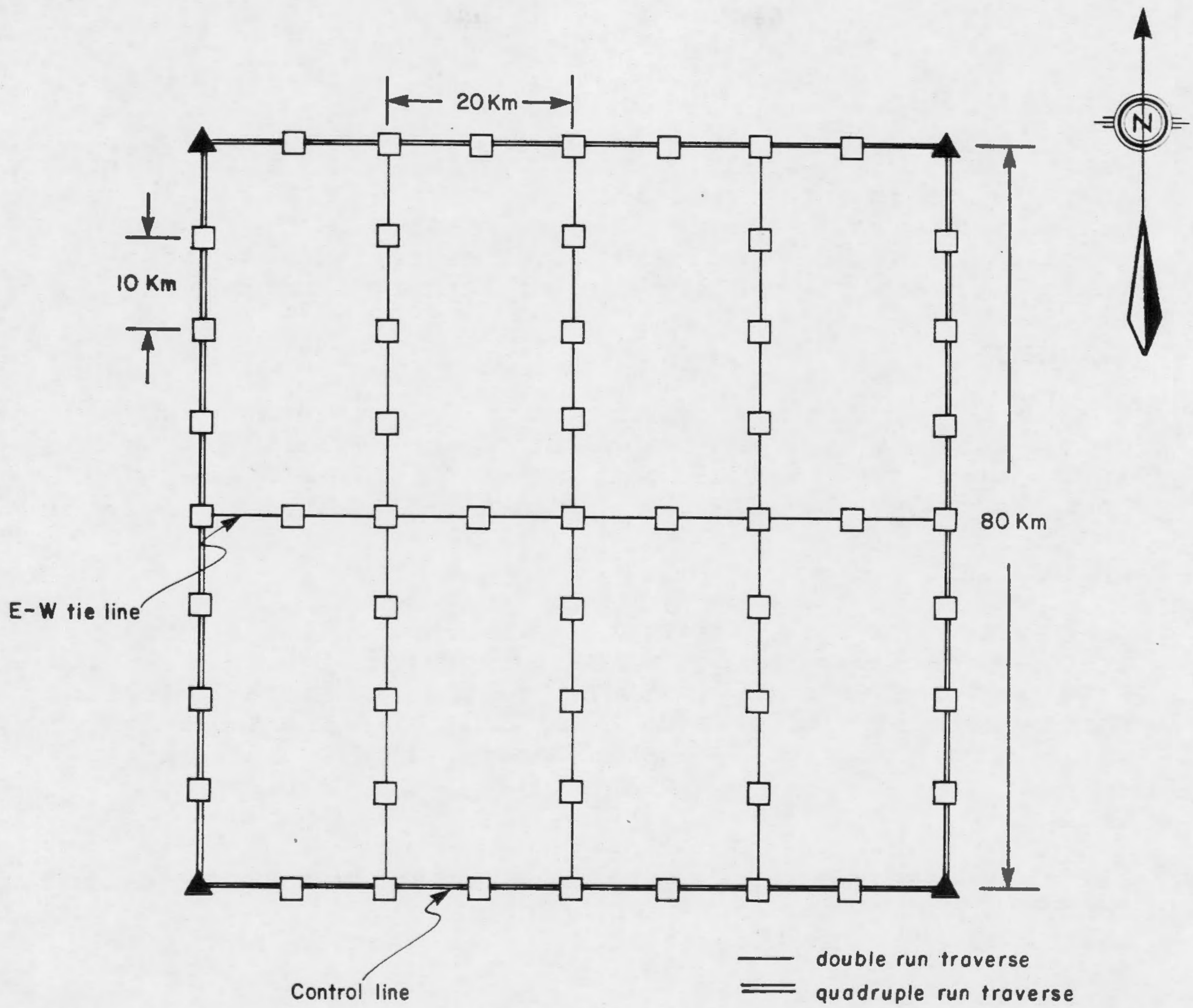


Figure 5-8 - Inertial Traverse Network

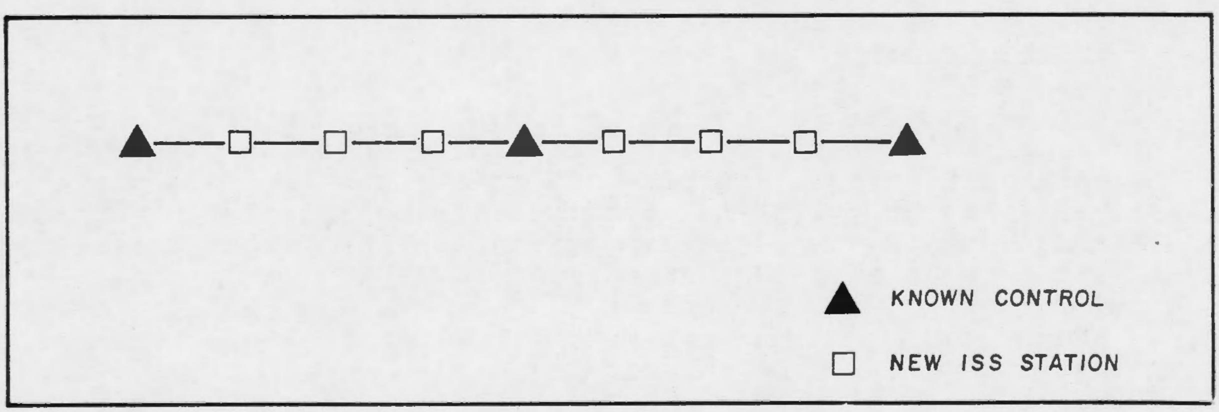


Figure 5-9 - Single Line Inertial Traverse

f) Time and Turning

When traversing, it is very important that time is not lost searching for stations as this will cause the measuring accuracy to deteriorate. The heading sensitivity of the equipment requires that the changes in vehicle direction on a traverse be kept to a minimum.

g) Alignment

A half hour minimum alignment must be completed each day before the daily traversing can start. This alignment must meet the specifications outline in the "ISS Operator's Manual and Guide" (Geodetic Survey, 1986).

h) Calibration

The system must be calibrated within the local working area for accelerometer scale and misalignment errors. This is accomplished by using control that is oriented North-South and East-West. For full details on dynamic calibration refer to (Geodetic Survey, 1986).

i) Control

All traverses must be controlled at each end by reliable Geodetic horizontal and vertical control, or by other ISS positions and elevations already established.

j) Other Considerations

More details on the following: alignments, calibrations, system station marking, track/range, running a traverse, etc. are available from Geodetic Survey.

k) Survey Design

A grid pattern of station spacing, indicated in Figure 5-8, is the standard design employed by Geodetic Survey when establishing multi-purpose ISS area control. This pattern meets the Topographical Survey Division's 1:50,000 mapping requirements when the mapping photography flight lines are flown in an east-west direction.

When a grid pattern is not required or is impractical, an alternative is a single-line traverse run in both directions (Figure 5-9). A known point near the centre of the traverse is essential to provide an independent check on results.

5.3.6 Operational Requirements

a) Personnel

Depending on the size of the project, the time of the year, the number of inertial systems and aircraft, and the geographic location, the crew size may very from 5 to 10 persons, including aircrew.

b) Station descriptions and coordinates for all new and existing stations must be available. Coordinates for new station should be scaled from 1:50,000 maps. To minimize the time lost on a traverse, all stations to be occupied should be targetted in advance to permit easy sighting from the air.

c) Any control point that is inaccessible by helicopter must have an accessible eccentric station established.

d) Bulk turbo fuel should be cached before field operations begin in isolated areas.

5.3.7 Daily ISS Procedures

Figure 5-10 illustrates the sequence of events of an ISS daily field survey operation. The recording forms required at each step are noted. Sample forms are included in Appendix D, pp.D8 to D12.

5.3.8 Field Computations

All field processing of ISS data is done using an HP-87 micro-computer. Field processing consists of the following:

a) The mission data is smoothed for each traverse, including editing of operator-entered information at each point. Both the raw and smoothed traverses are printed and stored on floppy disks.

b) The forward and back smoothed traverses are merged to determine whether the traverse differences are acceptable. Another program lists the station positions and elevations from the smoothed traverses, computes the overall means and the differences from the mean. This program is also used to determine data acceptability.

c) A nine-track copy tape of the raw data is made and verified in a format suitable for final processing. A "Pads Software Package",(Lawnikanis 1986),is available for documentation and instructions on the use of these programs.

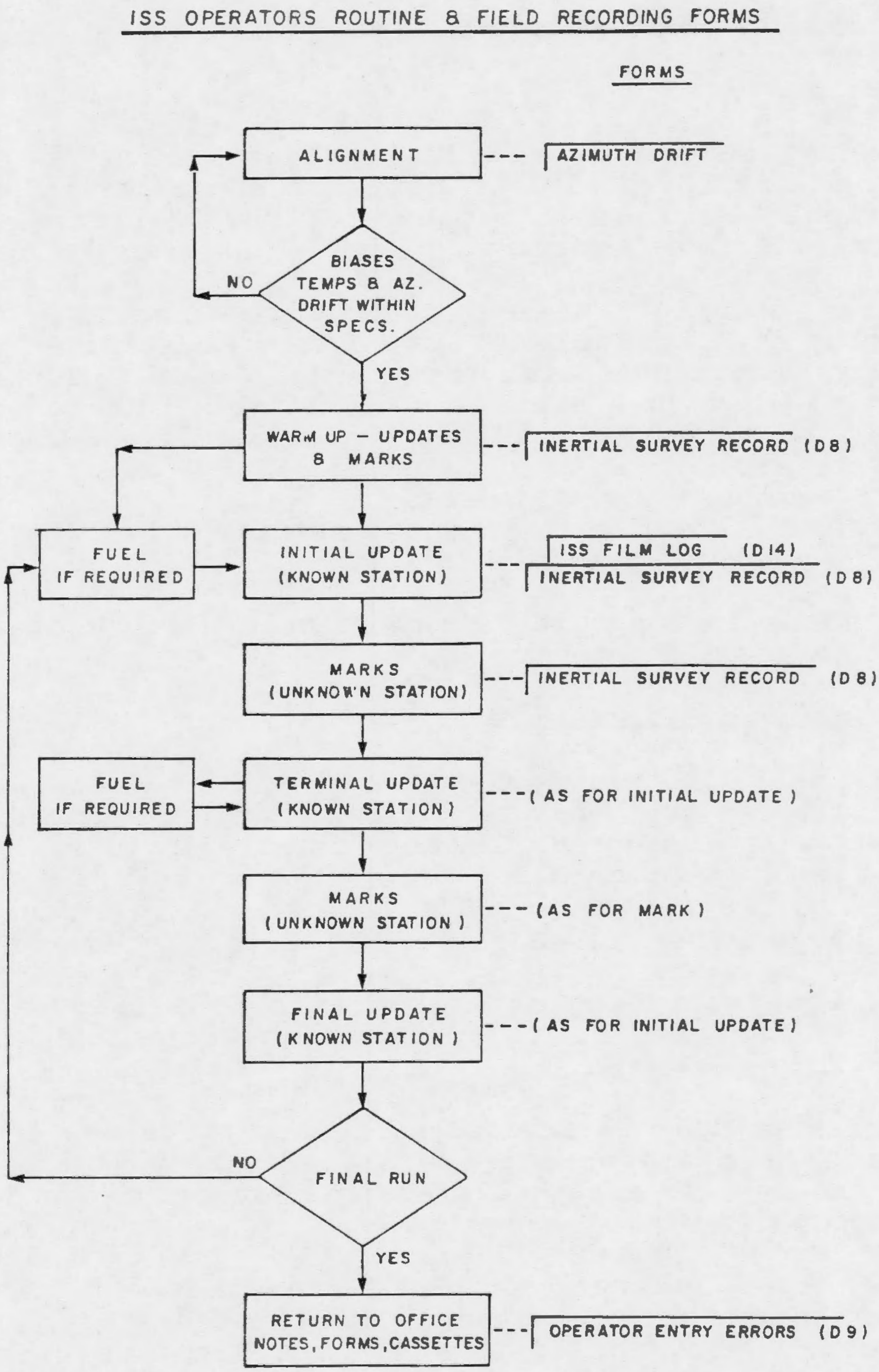


Figure 5-10

ISS Field Recording and Operating

5.3.9 Office Computations

Figure 5-11 illustrates the steps involved in the post-field adjustment and analysis of ISS data.

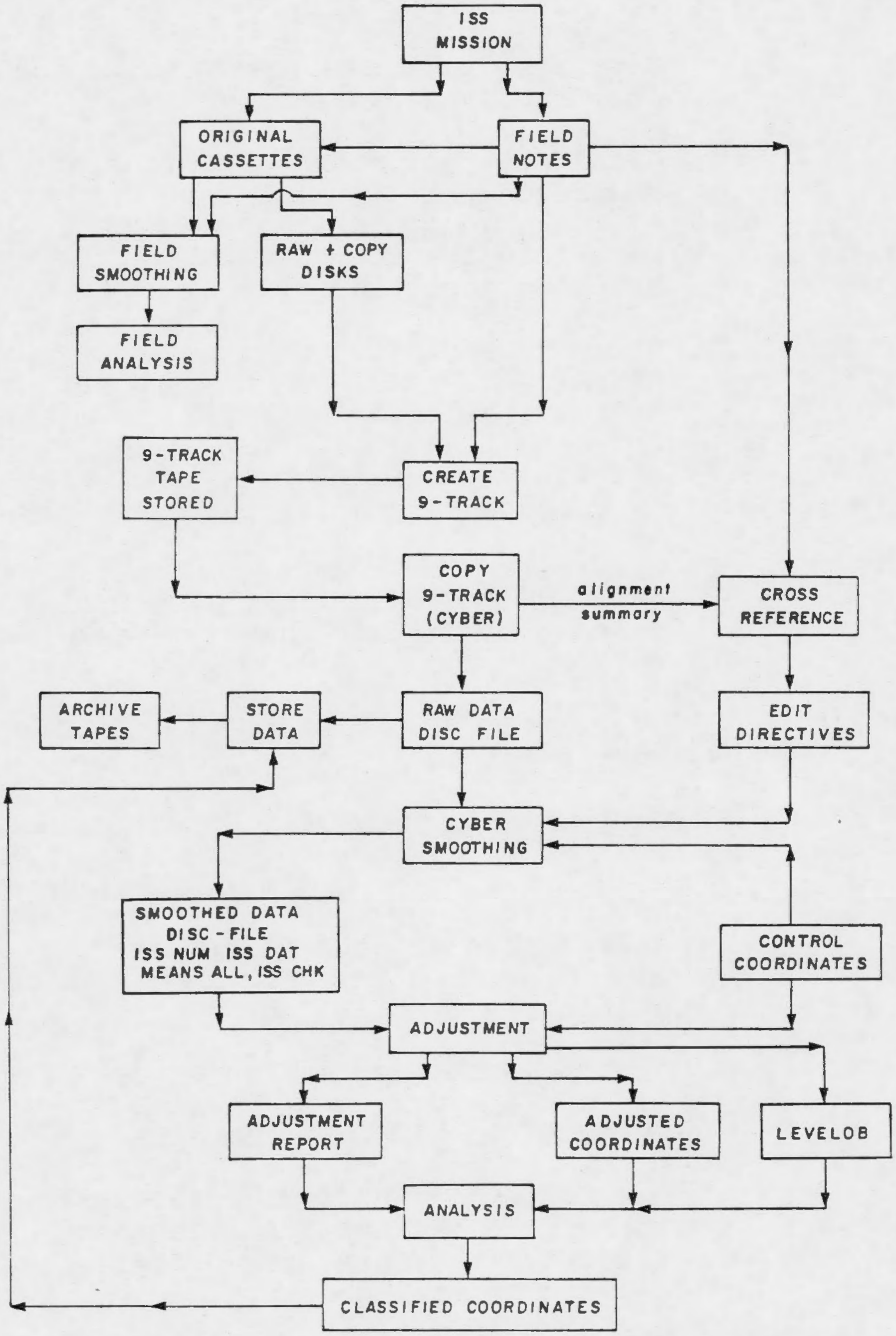


Figure 5-11 - ISS Data Processing

ORIGINAL CASSETTES	Data collected in the field are stored on raw data tapes during ISS missions.
FIELD NOTES	Raw data are smoothed in the field using the HP-87 computer.
COPY DISKS	Copy and raw disks are made in the field from raw data casssettes.
CREATE 9-TRACK 9-TRACK TAPE STORED	Data from copy disks are transferred in the field onto 9-track tape using the HP-87 computer. This tape is then shipped back from the field and stored in the computer tape library.
COPY 9-TRACK	From the 9-track tape, an initial "ISSDAT" is run on the computer using a program called "ISSDAT". This initial "ISSDAT" is only an alignment summary; traverses are listed by alignment identification.
CROSS REFERENCE	Traverses have not been adjusted at this stage. This program is useful as a preliminary data check on the 9-track data. "ISSDAT" is rerun smoothing the horizontal and vertical traverses. Output from "ISSDAT" contains all the information from an initial "ISSDAT" divided into individual traverses, horizontal and vertical. The traverses are checked and the necessary corrections are made e.g.wrong offset, incorrect station, etc.
EDIT DIRECTIVES	To aid in the correction of data from "ISSDAT", program "ISSCHK" is run. Output from "ISSCHK" contains: - a table listing individual marks and their deviations from the mean, - forward/back traverse differences, - known control checks, - other useful information.

SMOOTHED DATA DISC FILE	Final "ISSDAT" is run with the necessary corrections to the data.

Final smoothed traverses are stored on file and used as input to the program "ISSNUM".

"ISSNUM" is run replacing the five-digit STORE station numbers used in the field with the DATA unique data bank numbers.

Output from "ISSNUM" is an update file used as input to the free adjustment for the horizontal, and in "ISSPOS" for the vertical.

"ISSPOS" and "GANET" are used to run an initial free adjustment. The free adjustment points out:

1) errors in control,
2) poor traverses,
3) eccentric stations.

Should any correction have to be made, the free adjustment is rerun.

Corrections are made to the "ISSNUM" file which is the input to the free adjustment. The final ADJUSTMENT free adjustment is stored on file and is used as input coordinates for the MAY '76 and NAD'27 adjustments.

If the vertical adjustment is being done, program "ISSPOS" is used. The output from "ISSPOS" is used as input to "LEVELOB".

Final coordinates and elvations for the adjustment are collected from the Data Services Section.

Final fixed ajustments using "GANET" and "LEVELOB" are run on the computer. Catalogued files from these adjustments contain final coordinates.

5.3.10 System Maintenance

Maintenance consists of three steps:

a) determination of faults;

b) isolation of the fault to a Line Replaceable Unit (LRU) (i.e. cables, CDU, DPU, IMU, DSU, PSU);

c) further isolation of the fault in the LRU to broken wires, bad switches, bad circuit boards, etc.

While determining the fault, it is important to note the actual symptom and not to make assumptions. Problems in one area can manifest themselves in other areas.

Once the fault is isolated, LRUs may be changed until the problem is corrected. Attempts may then be made to pinpoint the fault within that unit. If circuit boards are involved, they are interchanged until the faulty board is identified. Minor repairs such as broken wires, bad switches, replacement of integrated and other solid state circuits can be done "in-house". Gyro and accelerometer replacement is carried out by the manufacturer.

5.4 **Aerodist**

Aerodist was used by the Surveys and Mapping Branch between 1962 and 1973. After 1965, it was used to establish primary control over an area of 1.6 million square kilometres. This corresponds to about 16% of the land area of Canada. An additional half million square km was controlled for mapping before 1965. Doppler Satellite Positioning replaced Aerodist in 1974.

5.4.1 Description

Aerodist is lightweight electronic distance measuring equipment capable of high relative accuracy and a range of about 140 km, aircraft to ground. The master equipment,carried in a DC-3 aircraft could measure to either two or three ground stations simultaneously. Aerodist was built by Tellurometer of South Africa and works on the same principle of phase comparison as the Tellurometer. In addition to the aerodist equipment, a Wild RC-8 aerial camera, an Air Profile Recorder (for heighting the aircraft and to establish ground elevation), automated meteorological equipment (to establish the refractive index of radio waves) were carried in the aircraft. The aircrew consisted of a pilot, co-pilot, navigator, air profile recorder operator and two aerodist operators.

5.4.2 Uses

Aerodist control was established for several purposes. Besides its use in mapping and for framework control, it was used to position navigational hazards, as well as aids to navigation such as Loran sites on the east coast and Decca sites in the north. Sable Island (240 kilometres off Nova Scotia) and Funk Island (100 kilometres off Newfoundland) were first accurately positioned using aerodist.

5.4.3 Method of Survey

A network of primary aerodist control consists of ground stations spaced in a rectangular grid pattern of 100-110 kilometres. The basic configuration is the double-braced quadrilateral made up of 6 ground stations with all 15 possible conncecting lines being measured. Six ground stations are manned at a time with an average 2 weeks stay at each station.

To measure a line, the DC-3 flies across the imaginary line joining the two involved stations while the distances between the aircraft and the two stations are continuously being recorded on a strip chart recorder. The sums of these two distances, if plotted would, approximate a parabola. The minimun distance is computed by least squares and the minimum sum is corrected for aircraft height, ground station ehight, eccentricity, refraction, curvature of the ray path and sea level arc distance. Six such crossings at different flying heights consitute one measurement group. A line measurement consists of a minimum of two measurement groups.

After establishing the framework control, additional horizontal control for mapping may be carried out by positioning the aircraft from measurements to three of the ground control stations, simultaneously with the exposing of vertical photography. The flight lines are run along each degree of longitude in the area. Using the corrected distances from the three known stations, a position for the nadir point of the aerial photograph may be computed and the positioned photograph used in a photogrammetrical aerotriangulation block adjustment.

5.4.4 Accuracy

Aerodist nets have an accuracy in the order of 10 ppm (1σ). Ground control derived from Aerodist controlled photography is accurate to ±5 metres relative to the control used.

5.5 **Azimuth Determination**

5.5.1 Introduction

The purpose and scope of this section is to describe procedures to be followed for the determination of second and lower-order azimuths using "The STAR ALMANAC for Land Surveyors" (SALS) as a reference for the astronomical data.

5.5.2 Azimuth determination for Second-Order Surveys

Second-order azimuth determinations are sometimes required on supplementary control surveys to orient specific lines or to control the orientation of traverses and other survey networks. Laplace observations are not dealt with in this section; however, Geodetic Survey of Canada will provide surveyors with advice on precise astronomic determinations if required.

Two techniques are described:

- Determination of astronomcial azimuth with a theodolite; and
- determination of azimuth, from astronomical north, with a gyro-theodolite.

Types of surveys for which azimuths or orientation are required:

- traverses for mapping,
- engineering surveys,
- hydrographic surveys,
- mining surveys,
- links between existing surveys,
- ties to reference, azimuth and eccentric markers,
- ties to boundary markers.
- orientation of radio telescopes,
- precise orientation for laboratory instruments, etc.

5.5.2.1 Determination of Astronomical Azimuth (Hour angle)

a) Instrumentation

- theodolite (1″) with accessories for day and night observations
- tripod
- elbow eyepiece
- stride level (if available)
- Roelofs prism attachment
- thermometer
- barometer
- tables (SALS)
- radio (for receiving WWV or CHU time signal)
- digital clock or chronometer (in Universal Time or Sidereal Time)

b) Observation Requirements and Accuracies

	STARS or	SUN
• Minimum number of sets	8	8 (4 in morning) (4 in afternoon)
• Expected standard deviation (above 60°N latitude)		
- internal	2″	3″
- external	4″	6″
• Expected standard deviation (above 60°N latitude)		
- internal	3″	4″-5″
- external	6″	8″-10″
• The position of the station must be known to a reasonable degree of accuracy. Geodetic coordinates (latitude and longitude) should be known to at least	±2″	±2″
• If the position of the station is not well-known, two stars must be observed, one east and the other west of the meridian at about the same altitude; the number of sets required is 4 for each star.		

Table 5-6 - Azimuth Observation Requirements and Accuracies

In the following documentation, "Celestial Object" (CO) refers to either the Stars or the Sun.

5.5.2.2 Observation Procedures

The observation procdures for "Polaris" or other bright stars, and for the Sun, at any hour angle are described below.

Experience has shown that the most favourable period for observations on stars and the reference object (RO) is during or just after the evening twilight when the RO appears seady for accurate pointings.

Observations on the Sun should be made between 8:00 and 10:30 a.m. or between 2:00 and 4:30 p.m., Local Apparent time. These periods will give a minimum error in the observed azimuth by hour angle. Surveyors should avoid observations when the sun is near the meridian.

a) Observation on Polaris

There are two methods which will help find Polaris depending on the observer's location and the time of day (daylight or darkness).

In Daylight

- The altitude of Polaris can be quickly computed by using the Pole Star Tables found in the SALS if the latitude of the station at the Local Sidereal Time (LST) are known. These tables give corrections which, when applied algebraically to the latitude of the sation, give the altitude of Polaris.

$$h = \phi - (a_o + a_1 + a_2) \qquad \text{(SALS)} \qquad (5\text{-}13)$$

Example:

Latitude of the occupied station:45°25'46" or 45°25'.8
Date: 1985 May 14

$LST = 15^h03^m40^s5$

From tables: a_0 = +47.04 = Correction for LST
+ a_1 = .00 = Correction for latitude
+ a_2 = + .25 = Correction for month

+47.29 or 47'.3

Therefore,

Altitude of Polaris (h) = 45°25.8 - (+47.3)
= 44°38.5 or 44°38'30"

or Zenith Distance (Z) = 90° - h = 45°21'30"

Polaris can be found by setting this angle on the vertical circle of the theodolite (h or Z) and pointing the telescope to the approximate North.

Approximate North can be obtained using a compass or a map. In high latitudes, it may be necessary to take a Sun azimuth to determine north in order to locate Polaris.

At Night

- Polaris can be located by projecting a line through the pointers of the Big Dipper (see Figure 5-12). It is the only bright star in that region of the sky.

  It is good practice to make a sketch in the field book showing the approximate directions of the CO and RO relative to north. (See Figure 5-13).

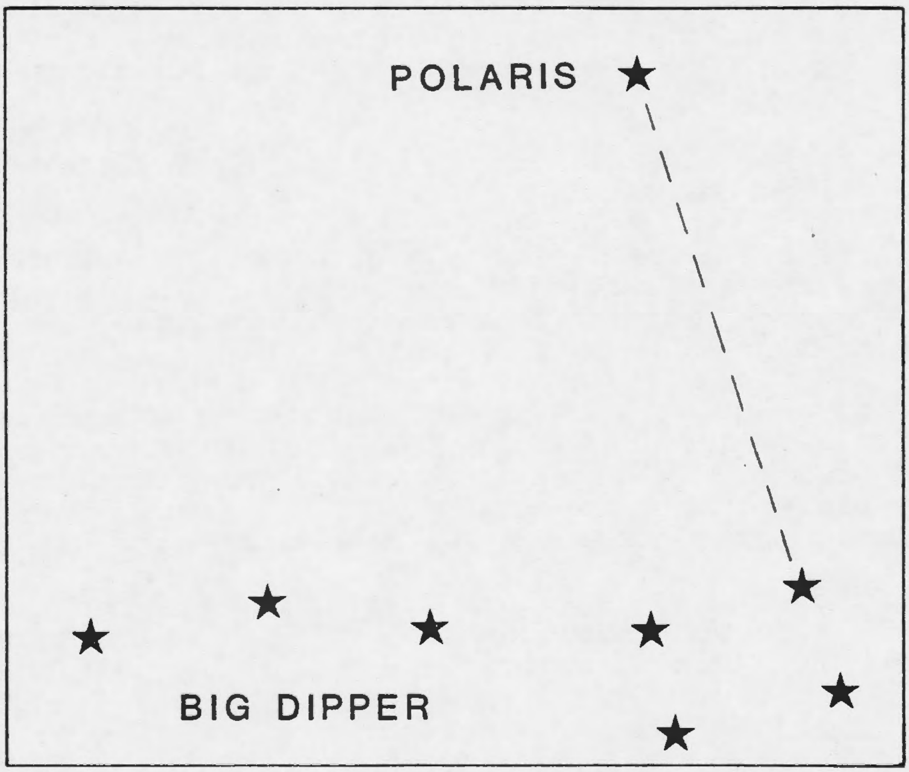


Figure 5-12 - Locating Polaris

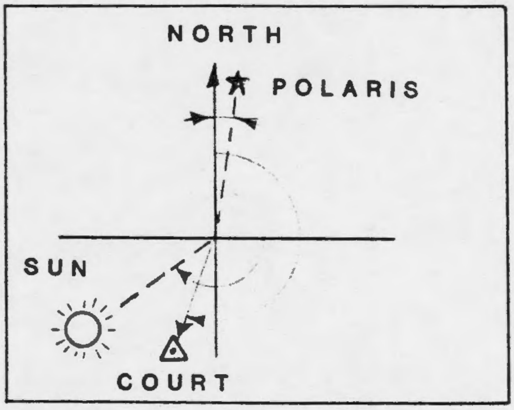


Figure 5-13 - Relationship of Celestial Object, Reference Object and North

b) Observations

1) Check the adjustment of the theodolite. Choose stable ground for the set-up or, if necessary, occupy an eccentric station. A stable instrument set-up is crucial to accurate azimuth determination. Position the tripod legs so as not to interfere with the observer. When occupying an eccentric station, measure the horizontal distance and the azimuth difference between the true and the eccentric markers. Redundant measurements must be made to isolate blunders.

2) If a stride level is not used, record any dislevelment of the plate-level.

3) Determine the rate of the digital clock or chronometer before and after the field observations (a comparison of the clock against the radio time signal should be recorded to 0.1 second of time).

4) Select an RO, preferably one kilometre or more away, to avoid having to refocus the instrument between pointings and so that the centering and pointing errors will not adversely affect the accuracy of the azimuth (one second of arc is equal to about 0.5 cm at a distance of 1 km).

5) Sight on the RO in left clamp (L). Set the circle to 0°00′ (for the first observation). Read and record the horizontal circle reading (HCR) and clamp position.

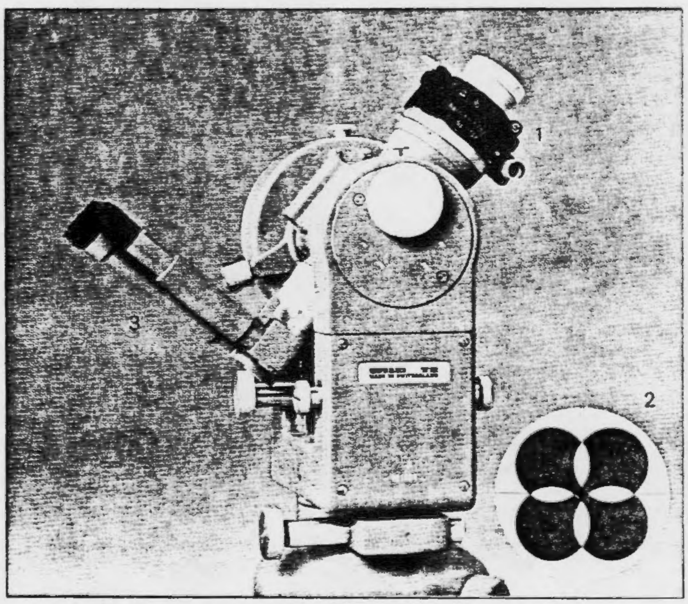
6) For STARS: Point the telescope at Polaris or another easily identifiable bright star and, turning both vertical and horizontal slow motion screws, bring the star to the centre of the crosswires of the telescope.

Using the stop watch method (described later), determine the time to the nearest 0.5 seconds for the pointing on Polaris. For stars with declination less than 60°, record the time to 0.2 seconds (an experienced observer can obtain the time of pointing with an accuracy of 0.2 seconds). Record the graduation readings for both ends of the stride level, and horizontal and vertical circle readings, (HCR) and (VCR).

Stop-Watch Method of Timekeeping

When the centre of the crosswires of the telescope is set directly on the star using the slow motion screws, a stop-watch is started. When the chronometer second hand is at a whole second mark the stop-watch is stopped and the chronometer time noted. The chronometer time minus the stop-watch interval will give the correct time.

For SUN: Using a Roelofs prism or a dark glass between the observer's eye and the instrument eyepiece, point the telescope at the Sun and centre the small diamond-shaped gap formed by the four overlapping images of the Sun (see Figure 5-14). At this instant record the time, accurate to 0.2 seconds, HCR and VCR, and the clamp position (Roelofs, 1948).



COURTESY OF WILD HEERBRUGG

Figure 5-14 - Wild T2 with Roelofs Solar Prism Attached

Wild T2 with Wild-Roelofs solar prism (1) and diagonal eyepieces (3). With the solar prism, four images of the Sun are seen (Insert 2). The small black diamond in the centre corresponds to the centre of the sun.

If a Roelofs prism is not available the Sun's upper and lower limb can be followed in one quadrant of the field of view with the horizontal crosswire, until the right or left limb reaches the vertical crosswire. At the instant when both the horizontal and vertical crosswires are tangent to the Sun (see Figure 5-15) record the time accurate to 0.2 seconds) and the position of both ends of the level and the HCR.

Indicate on a sketch the location of the Sun in the field of view of the instrument.

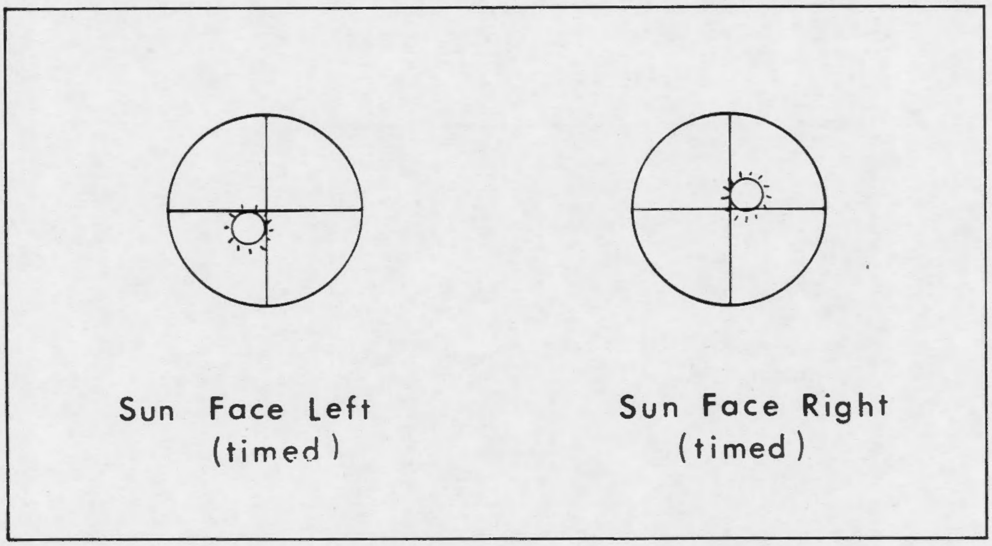


Figure 5-15 - Location of Sun in Field of View

7) Transit the telescope, point on the star and repeat step (6). If a Roelofs prism is not being used for sun shots, point on the Sun in the quadrant of the field of view that is diagonally opposite the quadrant previously used.

8) Repoint on the RO and record the HCR and the clamp setting.

This completes one set of observations for azimuth. For subsequent sets, change the horizontal circle setting and repeat the above procedure starting with step (5) with the theodolite in right clamp (R) position. The theodolite set-up must be re-checked for stability.

For Star observations, one azimuth determination comprises 8 continuous sets. For Sun observations, two determinations are preferable, one in the morning and one in the afternoon consisting of 4 sets each.

5.5.2.3 Computations and Formulae

The computations involved with each set of observations are:

a) mean the direct and reverse horizontal readings to the RO and to the celestial object (CO);

b) compute the angle from CO to RO;

c) determine the mean time of observation to compute the hour angle and to obtain the declination of the CO;

d) compute the azimuth of the CO;

e) when using a stride level a level correction must be applied to the azimuth of CO. The correction for the inclination of the horizontal axis is:

$$\frac{d}{4}\left[(w - w') + (e - e')\right] \mathrm{Tan}\ h \qquad (5\text{-}14)$$

(Hoskinson & Duerksen, 1947)

where

d = value of one division of the stride level (or plate level) in seconds of arc/division (for Kern DKM2A and Wild T2A plate level d = 20″)

h = altitude of the Star (or Sun)

w,w',e,e' = the readings of the west and east ends, respectively, of the stride level. The prime letters refer to the readings taken in the position in which the numbering increases towards the east. This level reading method is for stars near the meridan. (For CO near the prime vertical use a similar and systematic level reading method eg. the prime letters refer to the readings in which the numbering increases towards the south).

f) Using the angle determined in (b), compute the azimuth to the RO.

The azimuth of a Star or the Sun, at any hour angle, can be found using the formula:

$$\text{Tan } A = -\frac{\text{Sin } t}{\text{Cos } \phi \text{ Tan } \delta - \text{Sin } \phi \text{ Cos } t} \qquad (5\text{-}15)$$

where

A = azimuth of CO (if CO is east, azimuth = A or 180° - A; if CO is west, azimuth = 180° + A or 360° - A)

t = hour angle of CO

ϕ = latitude of observing station

δ = declination of CO

"t" can be found by either of two fundamental astronomical relationships, depending on the type of time used by the observer.

$$t = LST - RA$$
$$t = UT + E$$

where

LST = Local Sidereal Time

RA = Right Ascension of CO

UT = Universal Time (see note below)

E = Difference between the Greenwich Hour Angle of the Sun and Universal Time

NOTE: The tabulated values for E and the declination are based on Universal Time in the SALS. The standard radio time-signals give Coordinated Universal Time (UTC) and step adjustments are made as required (at 0 hours on January 1 or July 1) so that the difference between UTC and UT does not exceed 0.9 seconds. The correction to be made to the standard radio time signal (DUT1) may be determined from the signal. See below.

Code for the Transmission of DUTI (Ref: CCIR)[1]

A positive value of DUT1 will be indicated by emphasizing a number (n) of consecutive seconds markers following the minute marker from seconds markers one to seconds marker (n) inclusive; (n) being an integer from 1 to 7 inclusive.

$$DUT1 = (n \times 0.1)s \qquad (5\text{-}16)$$

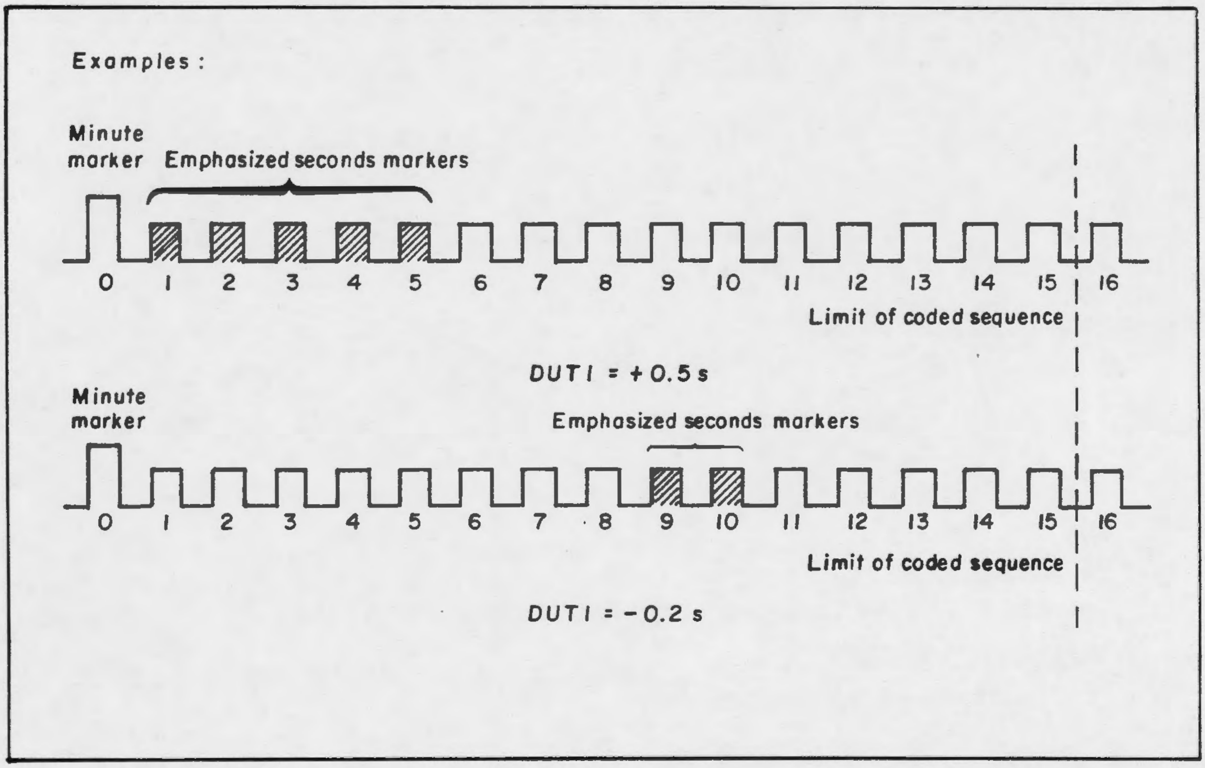


Figure 5-16 - Correction to UTC to Obtain UT

1 Consultative Committee of International Radio.

A negative value of DUT1 will be indicated by emphasizing a number (m) of consecutive seconds markers following the minute marker from seconds marker nine to seconds markers (8 + m) inclusive, (m) being an integer from 1 to 7 inclusive.

DUT1 = (m x 0.1)s

A zero value of DUT1 will be indicated by the absence of emphasized seconds markers.

The appropriate seconds markers may be emphasized, for example, by lengthening, doubling, splitting, or tone modulation of the normal seconds markers.

Example

The forms in Appendix E show:

- the steps to convert Local Mean Time to Local Sidereal Time and to compute the clock rate and level correction (E1);
- the azimuth reduction by the Hour Angle Method (Star) (E2);
- the azimuth reduction by the Hour Angle Method (Sun, Equation of time = E) (E3);
- the azimuth reduction by the Hour Angle Method (Sun, Right Ascension of mean Sun + 12h = R) (E4);
- the eccentric reduction and convergence - these two corrections are required to correct an eccentric azimuth (E5); and
- one page documentation for program STARAZ, available for the Texas Instrument (TI-59) pocket calculator, which can be used for office or field computations (E6).

5.5.2.4. Quick Azimuth Solution for Polaris using a Simple Algorithm

A quick check on the azimuth as computed in Appendix E2 can be obtained by using the Pole Star Tables found in the SALS.

The following is the algorithm found in the SALS tables.

Azimuth of Polaris = A = $(b_0 + b_1 + b_2)$ Secant (latitude) (5-17)

where

b_0 = correction for LST

b_1 = correction for latitude

b_2 = correction for month

This azimuth should be accurate to ±6 seconds of arc when corrections are properly applied.

Example. (using previous data in Appendix E2)

LST	$= 15^h23^m35^s2$ (argument for tables)
latitude	= 45°25′46″
b_0	= +16′.2 = 16′07″
b_1	= 0′.0 = 00′00″
b_2	= + 0′.1 = 00′06
SUM	= 16′.22 = 16′13″

A = 16′.22 X Sec 45°25′46″
= 16′.22 X 1.425
= 23′.11 = 23′07″
= 0°23′07″ (east of meridian)

This result is within 04″ of the previous result obtained with the azimuth by Hour Angle Method.

5.5.3 Lower order Azimuth (Atltitude Method)

Lower order azimuth determination is sometimes required to provide an approximate azimuth, needed to orient reference markers or eccentric stations, and in surveys which do not required second-order azimuth control. It is also used to provide a quick check for gross errors or blunders.

Instrumentation is the same as under section 5.5.2.1, with the exception of stride level and radio receiver. The latter may be replaced by any time service available inside the geographic location of the survey operations.

The observation requirements and accuracies, related to measurements shown under section 5.5.2.1, may be modified to:

- the minimum number of sets shown should be 4 for a third-order azimuth or 2 for a lower order azimuth

- the expected standard deviation will be two or three times larger.

5.5.3.1 Observation Procedure

The observing procedures, detailed under section 5.5.2.2, are to be followed. The time need only be observed to the nearest 0.5 seconds.

a) Observations

For the altitude method, with Polaris or any other bright star or the Sun, the following observing procedures should be followed.

1) Read and record the air temperature and barometric pressure (required for determination of the refraction correction to the altitude (h) or the zenith distance (Z) of the CO observed).

2) For SUN: As in step (6) of section 5.5.2.2, point on the small diamond-shaped gap formed by the four overlapping images of the Sun. At this instant, record the time, then read and record the HCR and VCR and the clamp position. If the Roelofs prism is not available the Sun's disc should be observed as described in section 5.5.2.2. At the instant when both the horizontal and vertical crosswires are tangent to the Sun, record the time and the circle readings.

NOTE: Indicate on a sketch the location of the Sun in the field of view of the instrument.

For STAR: as in step (6) of section 5.5.2.2, but the time may be recorded to half a second.

3) Transit the telescope, point on the CO and repeat step (2) above. If a Roelofs prism is not being used, point on the Sun in the quadrant of the field of view that is diagonally opposite the quadrant previously used.

4) Repoint on the RO and record the HCR and the clamp setting.

This completes one set of observations for azimuth. For subsequent sets, change the horizontal circle setting and repeat the procedure using the right clamp (R) position. The theodolite set-up must be re-checked for stability.

5.5.3.2 Computations

The computations involved with each set of observations are:

a) mean the direct and reverse horizontal readings to the RO and to the CO,

b) compute the angle from Sun to RO,

c) compute the zenith distance to the CO from the VC readings, correct for refraction (stars or Sun), and for parallax (Sun only),

d) determine the mean time (corrected time) of observation to obtain the declination of the CO,

e) compute the azimuth to the CO,

f) using the angle, determined in step (b), compute the azimuth to the RO.

The formula used to compute the azimuth of the Sun is

$$\text{Cos } A = \frac{\text{Sin } \delta - \text{Sin } \phi \text{ Cos } Z}{\text{Cos } \phi \text{ Sin } Z} \quad \text{(Military Engineering, 1976)} \qquad (5\text{-}18)$$

where

A = azimuth of the CO.
If Sun is east of the meridian, azimuth = A or 180° - A,
if it is west, azimuth = 180° + A or 360° - A..

δ = declination of the CO (positive if north of the equator, negative if south)

ϕ = latitude of the observing station

Z = zenith distance = 90° - h (Z is sometimes called co-altitude)

$$A_{RO} = A_{CO} + (HCR_{RO} - HCR_{CO})$$

5.5.4 General Information Related to the Computation

The declination "δ" can be obtained from SALS tabulated against UT. the latitude "ϕ" can be scaled from a 1:50,000 map of the National Topographic Series (NTS).

The zenith distance "Z" must be corrected for refraction (both Star and Sun) and parallax (for the Sun only). The refraction varies with the elevation angle, temperature and pressure; it always deflects the line of sight upwards which means that the refraction correction is subtracted from the altitude "h" or added to the zenith distance "Z".

The parallax correction is used to reduce the observations taken at the surface of the earth, to what they would be if taken at the centre of the earth. For this reason, the parallax correction is always subtracted from the zenith distance. The above corrections are found in SALS; however, for the parallax correction use 8".8 cos h, where h is the altitude of the Sun.

EXAMPLE:

The following forms are in Appendix E:

- the azimuth reduction by the Sun Altitude method (E7),
- one page documentation from program ALTITUDE METHOD which is available from the TI-59 (E8)

5.5.5 Identification of Stars

Once the altitude and azimuth of an unknown star have been observed, the time of observation noted and the latitude and longitude of the observing station are known, we can determine its right ascension (RA) and declination (δ) and from these, the star observed. The Atar Almanac for Land Surveyors contains all stars brighter than magnitude 4.5.

EXAMPLE

On 14 May 1985, a star of about magnitude 3 was observed at $5^h10^m10^s$ (EST) to have an altitude of 59°18'30" and an approximate azimuth of 299°55'45". The latitude and longitude of the observing station are N45°25'46" and W75°42'05" respectively. The Standard Meridian (Zone) of the place is 5^hW. The pressure was 1020 mb and the temperature -1°C. Find the RA and δ of the observed star.

IDENTIFICATION OF STAR

Observed altitude		=		59°18'30"
Refraction = 34" X 1.05		=		-36"
Corrected altitude	h	=		59°17'54"
Zenith distance	ξ	=	ZD =	0°42'06"
Azimuth	A	=		299°55'45"
Subtract 180° or (360° -	A)	=		360°
Zenith angle	Z	=		60°04'15"

Co-latitude = $(90^\circ - \phi)$ = ω = 44°34'14"

Hour angle = t

From $\cot t = \dfrac{\cot \zeta \sin \omega - \cos \omega \cos Z}{\sin Z}$ t = 46°21'37"

t = $3^h 05^m 26^s$ (5-19)

(Mackie, 1985)

From $\cos p = \cos \zeta \sin \omega - \sin \zeta \sin \omega \cos Z$

p = 37°41'32" (5-20)

To find the RA

EST of observation	=	5h10m10s
Zone	=	5
Corresponding UTC, May 14	=	10 10 10
R at 6h UT May 14	=	15 27 42
Gain in R in 4h10m10s = ΔR	=	40
GST = (UT + R + ΔR)	=	25 38 32
Longitude of observing station, W	=	5 02 48
LST at time of observation	=	20 35 44
Hour angle (t) of star	=	3 05 26
RIGHT ASCENSION, RA = (LST - t)	=	17 30 18

To find the δ

polar distance, p from above	=	37°41'32"
DECLINATION, $\delta = (90^\circ - p)$	=	N52°18'28"

On looking through the catalogue of stars (SALS), on pages 44-45, we find No. 478 β Draconis which has a right ascension of 17h30m08s and declination of N52°18'28" on the date concerned, and its magnitude is 3. There are no other stars in SALS which have coordinates near the calculated values, so β Draconis was the star observed.

5.5.6 Comparison Between Polaris and Sun Azimuth Methods

In this section, the examples were taken from actual observations that were made in the vicinity of Parliament Hill, Ottawa, Ontario. Table 5-7 compares the results of different methods.

DATE	METHOD	SETS	ORDER	OCCUPIED STATION	RO STATION	ECCENTRIC CORRECTION	OBSERVED AZIMUTH	FINAL AZIMUTH	STANDARD DEVIATION (INTERNAL)
MAY 13-14, 1985	POLARIS (AT ANY HOUR ANGLE)	16	2	NEPEAN-2 (ECCENTRIC)	COURT	+1°54'35.9"	208°53'40.3"	210°47'16.2"	0.3"
	SIMULTANEOUS OBSERVATIONS								
JAN. 29 1985	Sun (Hour Angle Method) and	6	3	NEPEAN-2	COURT	NIL	210°47'28.8	210°47'28.8	2.1
	Sun Altitude Method	6					210°47'01.3	210°47'01.3	2.6

Table 5-7 - Comparison of Azimuth by Polaris and Sun

5.5.7 Azimuth Determination using Gyro-Theodolites

5.5.7.1 Gyro Instruments

While there are a number of gyro-theodolites and gyro attachments available, this section will be restricted to comments based on experiences with the Wild GAK1 attachment and the MOM GiB1 gyro-theodolite. The GAK1 unit is quite small and light and can be mounted on top of a standard Wild theodolite such as the T1A, T16 or T2. The theodolite is modified at the factory by the permanent mounting of a bridge to which the gyro can be attached. The GiB1 is a considerably larger instrument with the gyro pod attached below the theodolite. Both instruments have similar operating instructions and yield similar results. For more details on the operating procedures, the reader is referred to the appropriate factory manuals and to (Gregerson, 1976).

5.5.7.2 Gyroscopic Azimuths

The determination of solar and stellar azimuths requires clear skies and knowledge of precise time during the period of observations. Gyroscopic azimuth observations are not weather dependent provided the RO can be sighted. They may be determined at any time of the day or night, and do not require time signals.

A gyro azimuth transferred to a line between two stations can be determined to an accuracy of ±20″ in about 20 minutes of working time using normal techniques, as outlined by the gyro manufacturer. The internal precision of a gyro azimuth determination can be reduced to the order of one or two arc-seconds by using special techniques and improved mathematical solutions. About 6 to 8 hours of observations are required to attain second-order standards.

The azimuth produced from gyro instrumentation is astronomic and, unless the astronomic longitude is known for the point of observation, no accurate comparison can be made to the geodetic azimuth. Estimating the prime vertical component of the local defelction does not necessarily ensure second-order accuracy of the azimuth, especially in mountainous terrain where large deflections can be expected.

5.5.7.3 Gyro Observing Methods

The gyro motor, which is suspended on a thin metal tape, hangs like a plumb bob and oscillates about the meridian plane. By measuring the amplitudes of the swing periods of these oscillations, the north direction can be found as the mid-position of the oscillations. The two basic gyro observing methods are the transit and the reversal point.

In the transit method, the theodolite's telescope remains clamped in the approximate north direction. A stopwatch is used to take the time of each transit of the gyro mark through the middle point of the scale. The angular correction, which must be applied to the provisional orientation, is proportional to the amplitude and the difference in the times taken to make the half swings to the east and west of the middle of the scale.

In the reversal point method, the oscillations of the gyro are followed by turning the theodolite's horizontal drive screw. At the reversal point (or elongation) of the oscillation, the gyro appears to be at a standstill for a few seconds. The horizontal circle reading for the reverse point is taken for the GAK1. For the GiB1 the amplitude scale reading is recorded, then the horizontal drive is turned to follow the gyro in the opposite direction. From the circle (or scale) readings taken at several successive reversal points the horizontal circle reading value of the north direction can be derived by using the "Schuler's mean" method (Gregerson, 1976).

Both methods give results of equal accuracy for low order azimuths (±20"). For higher order determinations the reversal point method is preferred, when the observations are taken visually, but the transit method can also produce equally good results if a self-recording electronic timing device is employed.

5.5.7.4 <u>The E-Factor</u>

The E-factor is the angular difference which exists between the optical axes of the theodolite telescope and the gyroscope scale reading system. This constant may undergo changes if an adjustment is made to the instrument or if the instrument is subjected to rough transport. The E-factor can only be determined accurately by observing lines of known astronomic azimuth.

Second-order astronomic azimuths for gyro-theodolite calibration have been established on the provincial EDM calibration baselines at Calgary, Edmonton, Grand Prairie, Lethbridge, Yellowknife, Winnipeg and Mississauga.

5.5.7.5 <u>Station Set-up</u>

The gyro-theodolite is rugged in the sense that it stands up well under both cold and warm temperatures (operational range is approximately -40°C to +50°C). However, it is a precise instrument, much more complex than most optical/mechanical surveying instruments. According to one gyro authority there are about 90 different error sources known for a gyroscope. Considerable care must be exercised in the transport and operation of the gyro-theodolite. It should be emphasized that the care taken in the set-up of the instrument is crucial to the successful observation of a gyro azimuth.

The solid set-up of the tripod is essential. If the ground is not stable, a solid base of some sort should be constructed - concrete blocks rocks cemented together, or stakes hammered into the ground to support the tripod legs. A catwalk around the station may be useful as well. An observation tent should be erected over the instrument to protect it from the effects of the wind and differential heating due to sunlight.

The gyro must be allowed to spin and swing for one full hour prior to any observations to attain a heat equilibrium within the gyro mechanism.

5.5.7.6 Expected Accuracies

For a gyro azimuth determination, a single set consists of observing 10 elongations or reversal points (5 swing periods) with the gyro motor spinning, then 8 elongations (4 periods) in non-spinning swing, plus direct and reverse pointings on the reference object. The first two elongations of the spinning-swing are omitted from the claculations. It requires about 50 minutes of time to make a single set at mid-latitudes. Higher latitudes requires more time.

Three sets are sufficient to control surveys for mapping purposes, while six sets on each of two days are required to attain second-order azimuth accuracy. The difference between the mean values for each day should not exceed 2 to 3 seconds of arc.

While the internal precision of such a gyro determined azimuth is in the order of 1 or 2 seconds of arc, as reflected by the standard deviation of the mean of the sets, the ultimate accuracy of the azimuth depends on the deflection of the plumbline at the point of observation. Therefore, because this will not be known in most cases, no attempt is made here to quote accuracies. The standard deviation of a gyro azimuth transferred to a line between two stations is in the range of ±3″ to ±10″ using special techniques, and 20″ using normal techniques. Gyro azimuth accuracies are known to deteriorate as the observer's latitude increases, and observations are not generally recommended above 78°N.

5.5.8 Compasses

5.5.8.1 Magnetic Compass

There are three major classes:

- pocket compass - generally hand-held and used for navigation or reconnaissance;
- surveyor's compass - usually mounted on a tripod and used primarily in forestry surveys and in retracing early land surveys;
- transit compass - mounted on an engineer's transit and used to provide orientation and a rough check on observed horizontal angles.

A compass azimuth refers to magnetic North, and must be corrected for the magnetic declination to refer the azimuth to true North. The declination varies with location (approximately fifty degrees of variation between the east and west coasts of Canada) and can be obtained from the National Topographic Series 1:50 000 or 1:25 000 maps.

Compasses provide approximate orientation on surveys where no other means are available. Azimuth by magnetic compass is only accurate to a few degrees of arc, providing only very approximate orientation.

Nearby ferrous metal objects and ore bodies or elctrical power transmission lines may cause large local deflections of the compass needle.

The magnetic compass becomes unreliable at high latitudes (near the magnetic pole) and is seldom used above about 60°N.

5.5.8.2 Astronomic Compass

The astronomic compass is used as a navigational tool, primarily in areas where magnetic compasses are unreliable. Sightings are taken on a CO (stars, Sun, moon, etc.) to determine the direction of astronomic North. Astronomic compassses are usually mounted in an aircraft but may be used on the ground.

To use an astronomic compass, the observer must know the UT, formerly Greenwich Mean Time (GMT), and his approximate latitude and longitude. The following steps are followed to determine North:

- the date and UT are used to determine the declination and Greenwich Hour Angle (GHA) of the celestial body from an almanac;

- the GHA of the body to be sighted is converted to Local Hour Angle (LHA) by applying the observer's longitude;

- the declination and LHA of the celestial body and the latitude of the observer are entered on graduated circles on the astro compass;

- the compass is levelled in its mount and the celestial body is sighted. The compass is so constructed that its base will then be oriented with respect to North.

5.5.8.3 Directional Gyro

A calibrated gyro provide a means of monitoring orientation changes. Its basic component is a vertically mounted rotor spinning about the horizontal axis at approximately 12,000 rpm. The inertial properties of the gyro are such that it will align its spin axis with North, thus providing a reference direction for navigation. For accurate navigation the directional gyro must be calibrated frequently to minimize the small precessional error or creep that accumulates in the gyro heading. This creep is due to a combination of mechanical friction in the gyro and the earth's rotation.

5.6 **Global Positioning System**

5.6.1 Introduction

The NAVSTAR (Navigation Satellite Time And Ranging) Global Positioning System (GPS) is a worldwide network of orbiting satellites. Precise position, velocity and time can be determined when the data from four satellites is received simultaneously, recorded and processed.

The first GPS satellite was lauched in 1978. Currently there are 7 satellites orbiting 20,200 kilometres above the earth. The full constellation will comprise 18 satellites plus three active spares, three in each of six equally spaced orbital planes, and having an orbit time of 11 hours 58 minutes. Up to ten satellites will be above an observer's horizon simultaneously. A satellite passing through the observers zenith will be 'visible' for about five hours.

5.6.2 Principles

Observations of the NAVSTAR GPS satellites can be processed to determine three-dimensional coordinate differences. These Cartesian coordinates (X, Y, Z) can be transformed into ellipsoidal coordinates. (ϕ, λ, h).

Each satellite contains a high precision oscillator with a fundamental frequency of 10.23 MHz. The four transmitted frequencies (L_1, L_2, C/A and P-code) are derived from this frequency. The carrier frequencies lie near the radio frequency L-band which extends from 390 to 1550 MHz. The L_1 carrier code (1575.42 MHz) is modulated with the C/A code (1.023 MHz) and the P-code (10.23 Mhz). The L_2 carrier code (1227.60 MHz) is modulated with the P-code. The message, containing the ephemeris data and clock correction parameters for the satellite, is transmitted at a rate of 50 bits per second. One message is 1500 bits long corresponding to 30

seconds of time. Satellite ephemerides are updated every hour with information derived from four tracking stations in California, Guam, Alaska and Hawaii.
The observations from two or more GPS receivers, simultaneously recording signals from the same four satellites, can be processed to obtain the difference in their positions in three dimensions and an accurate value for time. If one receiver is on a known position, the three dimensional positions of the other stations may be determined.

Basically, the receiver set measures the exact time the satellite signal is received and computes the time difference between transmission and reception. This time, multiplied by the speed of light, gives the distance from each of the four satellites being tracked. These four distances will give the receivers a three-dimensional position, time and velocity (if any).

5.6.3 Instrumentation

There are several receivers on the market which are capable of high precision positioning. Receivers such as the Texas Instrument - TI-4100 receive both L-band frequencies with both the C/A and P-codes. The Wild Magnavox - WM101, Litton Aero Products - LGSS, Trimble Navigator - 4000S, Rockwell Int. Collins-Navcore 1 receive the L_1 frequency with C/A code. Other receivers such as the GPS Sources Ltd. - GPS Land Surveyor #1991, Macrometrics Inc. - Macrometer V1000, Istac Inc. - 2002 and Magnavox - Land Surveyor are "codeless". The "codeless" receivers differ from the other types in that they do not require knowledge of the C/A or P-codes for normal full accuracy operation. The codeless receivers do not record the ephemeris data which must be obtained by other means.

5.6.4 Field Procedures

All GPS positioning carried out by the Geodetic Survey of Canada is done using differential positioning (relative positioning) techniques. That is, the determination of relative coordinates between two or more receivers simultaneously tracking the same satellite signals.

Minimum requirements:

1. The antenna must be plumbed over the survey marker and the height of the antenna phase centre above the marker measured to ±3 mm.

2. One set of meteorological observations must be made at the start and end of each session at each station of the group being measured. Information is to consist of dry bulb temperature to ±0.5°C, wet bulb temperature to ±0.5°C (or relative humidity to 5%) and atmospheric pressure to ±0.01 millibars (mb).

3. Each observing session must include at least 30 minutes of simultaneous data collection. Four (4) different operational satellites must be observed within the session. The session may consist of two (2) fifteen (15) minute periods where a minimum of three (3) satellites are tracked simultaneously at all sites occupied during the session. At least one of the three satellites tracked must be a different satellite in the second 15-minute period.

4. The measurement data recording interval must be no greater than 30 seconds.

5. Baselines between all adjacent stations must be observed.

6. For single frequency receivers baseline lengths should be kept under 100 km.

7. Satellites to be observed should be chosen so that the satellite/receivers geometry is optimum. For maximum accuracy, the GDOP (Geometric Dilution Of Precision) or PDOP (Positional Dilution Of Precision) should vary from large to small. These values are obtained from the output of the "GPS - ALERTS PACKAGE/CANADIAN GEODETIC SURVEY". See Table 5-10 for possible accuracies.

5.6.4.1 Network Design

An excellent paper by Richard A. Snay (Snay, 1985) of the U.S. National Geodetic Survey entitled "Network Design Strategies Applicable to GPS Surveys using Three or Four Receivers" was presented at the AGU spring 1985 Congress.

Much of the following on network design has been abstracted from this paper.

An efficient and reliable network involves compromise. Efficiency suffers at the expense of redundancy.

The following examples will be limited to network loops or traverses using four GPS receivers where each station is occupied twice and no two stations are jointly occupied for more than one observing session.

The number of observing sessions would then equal the total number of stations to be observed divided by 2.

If no line is reobserved then the network contains

$$\frac{(r - 1)nm}{2}$$ distinct directly observed lines

where r = no. of receivers

n = no. of occupations per station

m = no. of stations in network

Sample Observing Schedules (Snay, 1985)

$r = 4$, $n = 2$, all lines measured once

Observing Session	m = 6	m = 12	m = 13
1	1,2,4,5	1,2,4,5	1,2,4,5
2	3,4,6,1	3,4,6,7	3,4,6,7
3	5,6,2,3	5,6,8,9	5,6,8,9
4		7,8,10,11	7,8,10,11
5		9,10,12,1	9,10,12,13
6		11,12,2,3	11,12,1,6
7			13,1,3,8

Table 5-8 - GPS Observing Scenario, Four Receivers

5.6.5 Data Processing

Processing of GPS data at the Geodetic Survey is done using the carrier beat phase observations from the satellites and receivers. These are the measurements of the phase difference obtained when the satellite carrier frequency (with modulation frequencies removed) is compared to the nominal carrier frequency of the receiver.

5.6.5.1 Differenced measurements (Scherrer, 1985)

GPS measurements can be differenced across receivers, across satellites, and across time. Although many combinations are possible, the present convention for GPS phase measurement differencing is to perform the differences in the above order, first across receivers, second across satellites, and third across time.

A single difference measurement (across receivers) is the instantaneous difference in phase of a received signal, measured by two receivers simultaneously observing one satellite.
A double difference measurement (across receivers and satellites) is obtained by differencing the single difference for one satellite with respect to the corresponding single difference for a chosen reference satellite.
A triple difference measurement (across receivers, satellites and time) is the difference between a double difference at one epoch of time and the same double difference at the previous epoch of time.

Data collected using WM101 receivers, is processed in the field using the manufacturer's "POPS" software on a compac 286 computer.

The receiver data is transferred from the cassette to the computer using a MEMTEC cassette terminal model 5450XL.

The data preprocessing computes single point positions and baselines using double differences of baseline measurements.

A network adjustment of up to 10 stations can be done using the POPS software.

The satellite datum results (WGS84) are transformed to latitude, longitude and ellipsoid height.

The ellipsoid height will be converted to orthometric height using the geoid-ellipsoid difference from a model such as Rapp '78, augmented using available astro geodetic or gravimetric data.

Processing will include all [n(n-1)/2] baselines or n-1 coordinate differences computed simultaneously for all the data of each session, where n is the number of stations occupied simultaneously in a session.

5.6.6 Accuracies

The ultimate accuracy of determining one position with respect to another depends on many factors:

- number of satellites tracked simultaneously and their geometry (GDOP, PDOP)
- duration of tracking time
- collection rate of data

- azimuth and elevation angle of satellites tracked
- reception of reflected signals (multipath error)
- modelling trophospheric refraction
- modelling of ionospheric refraction
- receiver noise
- receiver clock error
- accuracy of satellite ephemeris
- satellite clock error
- earth's rotation

The major GPS error sources are shown in Table 5-9 (Brown et al, 1983).
Table 5-10 shows the position accuracy attainable for delta ranges accumulated over 0.5 to 2.0 hours based on the error budget in Table 5-9 (Brown, Sturza, 1983).
Figure 5-17 (Scherrer, 1985) indicates accuracy estimates for different geodetic techniques of measurement at different distances.

ERROR SOURCE	MODEL	CONTRIBUTION TO ADR ** MEASUREMENT ERROR
Receiver Noise	White noise	0.32 mm 1σ
Receiver Clock	Random Walk 0.185 mm/√sec + White Noise	0.67 mm 1σ
Ionospheric - Dual Frequency Compensation	White Noise	0.98 mm 1 σ
Tropospheric	Range Error Markov 1σ = 10 mm τ = 2 hr	*τ 1 hr 0.17√τ mm τ 1 hr 10 mm 1σ
Satellite Ephemeris	Bias Along Track 0.8 m 1σ Radial 6.3 m 1σ Cross Track 3.0 m 1σ	*≈7√τ mm
Satellite Clock	Random Walk	*2 τ mm

* Only included in measurement noise model for point positioning when not estimated as stated.

** ADR is Accumulated Delta Ranges

Table 5-9 - Major GPS Error Sources

Time interval (Hr)	0.5	1.0	1.5	2.0
Average PDOP	74	20	10	6
Point Positioning Measurement Variance	0.389 m	0.550 m	0.661 m	0.764 m
Point Positioning RSS Position Accuracy	28.8 m	11.0 m	6.6 m	4.6 m
Translocation Measurement Variance	0.007 m	0.01 m	0.01 m	0.01 m
Translocation RSS Position Accuracy	0.5m	0.2 m	0.1 m	0.06 m

Table 5-10 - Point Positioning and Translocation Accuracies with one set of GPS Measurements

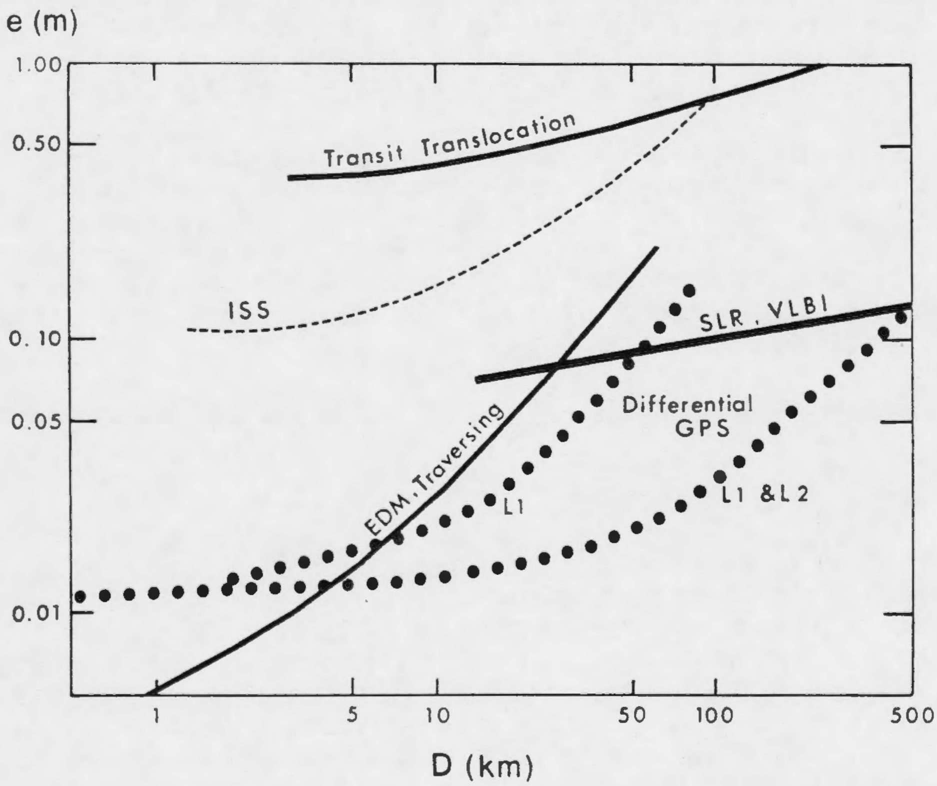


Figure 5-17 - Accuracy of Geodetic Techniques

5.7 TOTAL STATION OPERATION

5.7.1 Introduction

The Geodetic Survey has used 'total station' instrumentation for several years to make horizontal and vertical checks on 1:2000 scale Flood Risk Area maps in Manitoba.

'Total Station' is the name of a family of instruments which combine a theodolite and electronic measuring capability to give a simultaneous display of angles and distances. Various add-on units are available for on-site data processing, storage and printing. Most systems include a microprocessor which can be programmed to perform a variety of survey tasks and data manipulations.

5.7.2 Instrumentation

A typical 'total station' combination might consist of a one-second reading theodolite with 32x telescope, an infra-red distance measuring capability with a range of one to five kilometers, depending on the number of prisms used and the observing conditions, and a data terminal.

The data terminal contains both the microprocessor and data recorder in a weatherproof case. Memory size varies from 32K to 256K and is generally non-volatile magnetic bubble memory. Data stored on the data terminal is transferred to floppy disks for permanent storage or further manipulation using a micro-computer. Alternatively the data may be printed directly from the data terminal.

5.7.3 Procedures for Field Checking Flood Risk Area Mapping

The objective is to verify the horizontal and vertical integrity of 1:2000 maps.

Before field verification begins, coordinates of the required points are digitized from the maps and stored on 9 track tape together with their map elevations. These values are then transferred to the micro-computer and stored on floppy disks for use in the field office.

The coordinates, known elevations and identification numbers of check points chosen in the field, as well as those of control stations to be occupied as instrument stations, may be manually stored in the total station data terminal.

The total station is set up at a control station and levelled. Power is applied to the system and the program used to control the flow of data is initiated.

The initial set-up not only involves levelling the instruments but also informing the system of the height above ground of the instrument and the height of the prisms being used at any particular target station. Check shots to known elevations are carried out before and after any observations to ensure against blunders. (See Field Specs in 5.7.5.)

The observer establishes a reference azimuth between the set-up station and the R.O. station, whose co-ordinates are also found in the data terminal, by sighting the R.O. and entering the station numbers. The data terminal microprocessor now searches for information on each station and calculates and presets the grid azimuth and distance to the R.O. from the set-up station. The operator next enters the target or check station number, and the azimuth and distance to the target station are automatically calculated and set on the instrument. The rodman, guided by radio instructions, is directed along the azimuth the prescribed distance, and directs the pole-mounted prism back toward the observer for a check distance. Once the observer is satisfied that the rodman is within 1 metre of the intended check point the final angle and distance are measured in the direct scope position and this data is transferred to the data terminal. The telescope is then plunged and the indirect vertical angle is measured and recorded. This completes the required measurements for the check point. The program now uses the average vertical or zenith angle to determine the ground elevation at the check station, and displays both the observed elevation and the known map elevation in separate windows.

These values are now recorded and the differences examined. The check point, if it passes specifications, warrants no further attention, but should the check fail the rodman would leave a stake at the point so that the same spot could be re-observed from another instrument station or levelled to conventionally.

The rodman is then given a compass bearing and distance to direct him/her to the next target station. Usually two or three rodmen may be employed at this task due to the large distances to be covered and the speed of operation of the total station system.

Observations continue until the day's work is complete, at which point the data is then transferred from the data terminal to the mico-computer for storage and processing.

5.7.4 Field Computations

The data may be processed with existing software using either an HP-87 or an HP-85 micro-computer along with various combinations of printers and storage devices depending upon hardware availability.

Field processing consists of:

a) Transfer of data from the data terminal to personal computer for storage on disk or tape depending on hardware.

b) Field processing of daily data using program "PRCESS".This program name may change depending on the available hardware being used for data processing.

c) Final tabulation and analysis of daily data.

Note: Processing of data in the field is all that is necessary for this project. No further post-processing is required in the office.

5.7.5 Guidelines and Field Specifications

Geodetic Survey has used a total station to check Flood Risk Area maps over a period of three field seasons. In that time certain standards and guidelines have been developed and adopted to provide a procedure which will result in a reliable, blunder-free check of the existing maps. Following are some general guidelines that should be adhered to regarding the use of total stations.

a) instrument operation must be checked and calibrated before being used in the field.

b) Programs used in the field such as the 'Field' program used in checking Flood Risk Area maps, must be checked and verified to ensure they are operating correctly.

6 *FIELD RETURNS*

6.1 Data Transmittal

The frequency and method of data transmittal to HQ will vary from one project to another. Factors such as infrequent mail service and lack of regular air supply schedules in some remote areas, etc., will influence data transmittal. For each project, details of data transmittal will be included in the field instructions to the party chief. All hard copy data will be accompanied by the Geodetic Survey form "Transmittal of Field Records". This is a complete record of data being shipped. A duplicate is to be kept in the field-office. Complete records of each shipment will be kept by the party chief; these will include the registration number of items sent by registered mail, waybill numbers for shipment by air express, etc., the date of shipping and name of carrier.

On receipt of data at HQ, the person receiving the shipment must verify that the shipment is complete and agrees with that described on the accompanying transmittal form.

For more information, see Geodetic Survey Management Directive 86-2, "Registration, Handling and Transmittal of Field Records".

6.2 Survey Station Information

For all new survey stations established by a survey crew, the National Geodetic Data Base forms NGDB-58 and NGDB-59 (Appendices D13, D14) must be completed. The Station List Form NGDB 86-1 (Appendix D16) must include information on all stations visited during the field season.

6.3 Accounts

1) A field officer must submit accounts to HQ at the end of each month. See Chapter 3 of "General Instructions for Field Parties", for accounting procedures.

2) A Geodetic Survey "Monthly Resources Status Report" must be submitted to HQ by the third working day of each month. This is normally transmitted via telephone.

3) Upon completion of a field project, all outstanding financial commitments must be completed and bank accounts closed. For details see Chapter 3, General Instructions For Field Officers.

6.4 **Project Reports**

1) As each project is completed in the field, a report is to be submitted to HQ indicating the objectives accomplished and costs incurred (Form GPMS 77-4 Rev. 83-11-14).

2) At the close of field operations, a report must be completed for each project (ref. Geodetic Survey Management Directive 80-8).

6.5 **Vehicle Reports**

When vehicles are returned to Technical Field Support Services (TFSS) in Hull, Quebec or Western TFSS in Calgary, Alberta, a Vehicle Condition Report, Form No. 1852 Rev. 1 (TFSS) must be submitted with the vehicle, and a duplicate filed with the Positional Control Section. (Ref. General Instructions for Field Parties 1982, 2-12).

6.6 **Equipment Reconciliation**

1) At the end of field operations equipment no longer required shall be returned to TFSS (Hull, Quebec).

2) All instruments such as cameras, theodolites, EDM equipment, etc., must be red-tagged if in need of repair. It is helpful to submit a written report detailing events or causes leading to the malfunction.

3) Equipment from the Geodetic Survey Electronics Laboratory should be returned immediately upon completion of field operations. All technical equipment should be accompanied by a status report.

FIGURES

FIGURES (cont'd)

TABLES

FORMULAE

APPENDICES

APPENDICES

PAGE

APPENDICES (Cont'd)

PAGE

APPENDIX E **AZIMUTH FORMS**

APPENDIX A

SPECIFICATIONS FOR TARGET IDENTIFICATION PHOTOGRAPHY

(REF. T.S., 1978)

PURPOSE: To obtain vertical aerial photography of targetted control points suitable for stereoscopic transfer to the vertical mapping photography.

SPECIFICATIONS

1. **SCALE**

The scale of the identification photography shall be no greater than 2.5 times the scale of the mapping photography as defined by Topographical Survey.

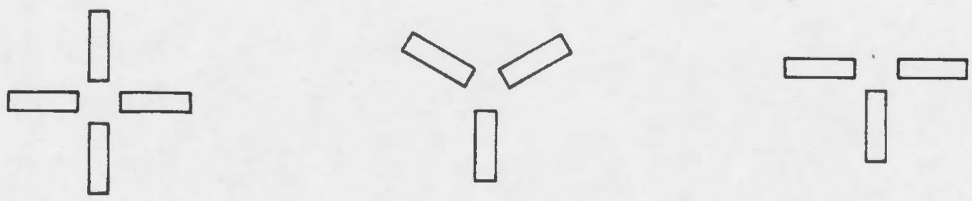
2. **TARGET SIZE**

The target size must produce an image 0.5 mm (minimum) in diameter on the identification photography.

The relationship between target size and identification photo scale is given in Appendix A3 (page A3-1).

3. **TARGET SHAPE**

3.1 The target must be one of the following distinctive shapes:



3.2 The ratio of the length of each arm to its width must be within the range 3:1 to 4:1.

4. **TARGET COLOUR**

4.1 The target shall be white.

4.2 The background must be chosen so as to provide contrast for the target. Sand or snow are unsuitable backgrounds and must be avoided, or artificially darkened in the vicinity of the target and the target width doubled.

5. **TARGET MATERIAL**

5.1 White cotton, white plastic or painted board are suitable target materials.

5.2 The target must be firmly retained in proper position until it is photographed.

5.3 If unusual conditions are encountered, and different target construction and identification can be demonstrated to be effective, these revised techniques should be approved in advance with the Topographical Survey Division.

6. **POSITIONING OF PHOTOGRAPHY**

6.1 The target image shall be in the centre of one photograph. The distance of this image from the centre of the photograph shall not be greater than 16% of the minimum format dimension.

i,e, 4 mm for 36 mm frames
10 mm for 70 mm frames

6.2 In addition to the above photograph, an overlapping (80%) sequence of photographs on either side of the target area is required. This run-up and over-run photography should extend at least two full frames on either side of the target. This will facilitate the transfer of the photo-identification to the mapping photography where detail is sparse.

7. **DIRECTION OF PHOTOGRAPHY**

7.1 The direction of photography should be chosen so that the run-up or over-run photographs record any prominent topographic feature in the vicinity of the target (lake, ridge, tree line).

8. **CAMERA**

8.1 A cartographic camera is preferred as it has a between-the-lens shutter and fiducial marks or reseau grid.

8.2 Non-cartographic cameras of 70 mm and 35 mm film sizes are acceptable (Vinten $1\frac{3}{4}$, Hasselblad 60 mm, Ricoh, Nikon, Pentax 28-35 mmm focal lengths).

8.3 The camera should have automatic film advance.

8.4 The camera focus shall be locked at infinity.

9. **MOUNTING**

9.1 The camera shall be vertically mounted in a shock absorbing mount (sponge rubber loaded to 1 lb./square inch provides adequate damping from aircraft vibrations).

9.2 Ideally there should be no intervening glass or plastic between the camera lens and the ground. If the photography is taken through a glass or plastic window, this window should be plane-parallel and at right angles to the optical axis of the camera.

9.3 Provision shall be made to level the camera so that the photography does not deviate from verticality by more than 10°.

10. **EXPOSURE**

10.1 The camera should be equipped with an automatic exposure control having shutter-speed priority. In lieu of this feature, an exposure meter must be used.

10.2 The shutter speed used shall not be slower than 1/500 second.

11. **FILM AND FILTER**

11.1 The film shall have a panchromatic black and white emulsion with adequate speed for short exposure times (i.e. Kodak Tri-X).

11.2 A filter, if used, shall be a minus blue haze cutting filter (i.e. Wratten 12).

12. **PHOTOGRAPHIC CONDITIONS**

12.1 The solar altitude shall be at least 20°.

12.2 The area surrounding the target (1000 m) must be free of cloud and cloud shadow.

12.3 Photography should be taken under clear conditions, however, photography under a uniform overcast is acceptable providing the film exposure is adequate.

13. ANNOTATION

13.1 Each negative of an identification strip shall be annotated with the station name or number, except in the case of 35 mm photography, where only each uncut strip of negatives of a target site need be so identified.

13.2 NORTH ARROWS shall be shown on each target identification photo.

14. FLIGHT RECORDS

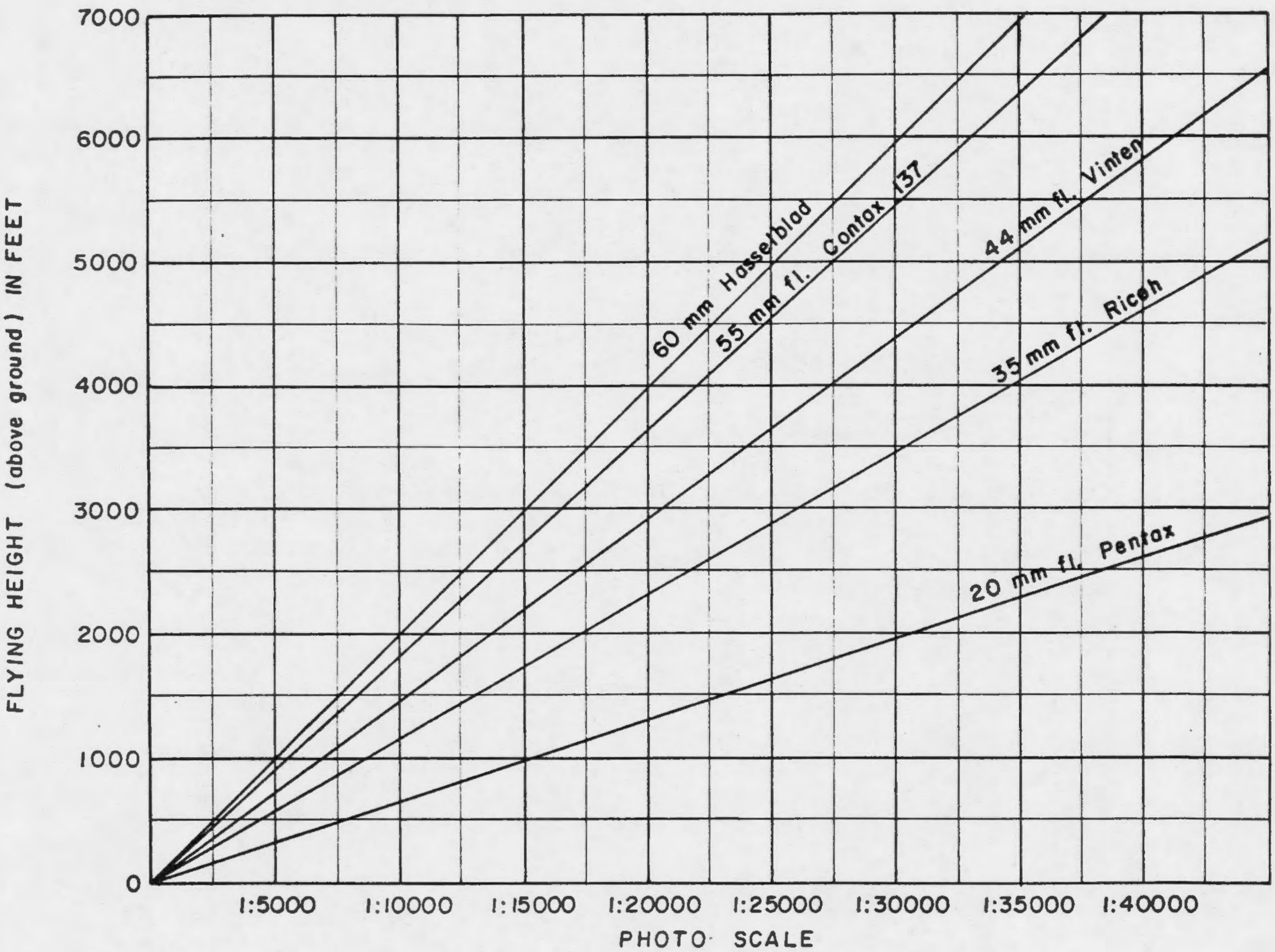
14.1 A flight log shall be supplied which provides the following information:

- Camera
- Focal Length
- Altitude above ground
- Date
- Station name
- Target shape

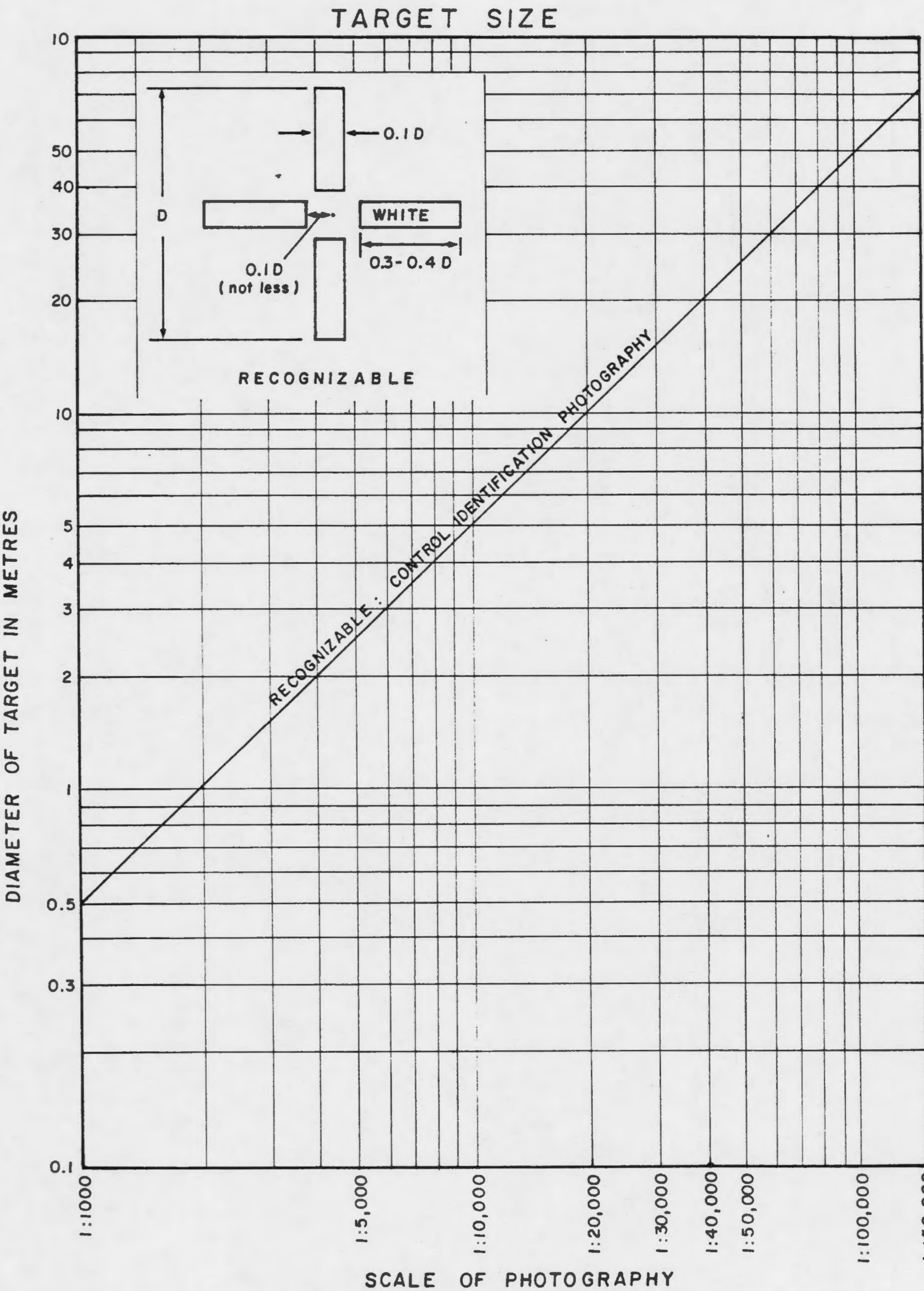
15. PHOTOGRAPHIC PRINTS

15.1 One set of contact prints is required.

15.2 The prints shall be sufficiently dark to provide a good target image.



DETERMINATION OF FLYING HEIGHT FOR VARIOUS CAMERAS


TARGET SIZE
0.1 D
D
WHITE
0.1 D
(not less)
0.3 - 0.4 D
RECOGNIZABLE
RECOGNIZABLE : CONTROL IDENTIFICATION PHOTOGRAPHY
DIAMETER OF TARGET IN METRES
10
50
40
30
20
10
5
4
3
2
1
0.5
0.3
0.1
1:1000
1:5,000
1:10,000
1:20,000
1:30,000
1:40,000
1:50,000
1:100,000
1:150,000
SCALE OF PHOTOGRAPHY

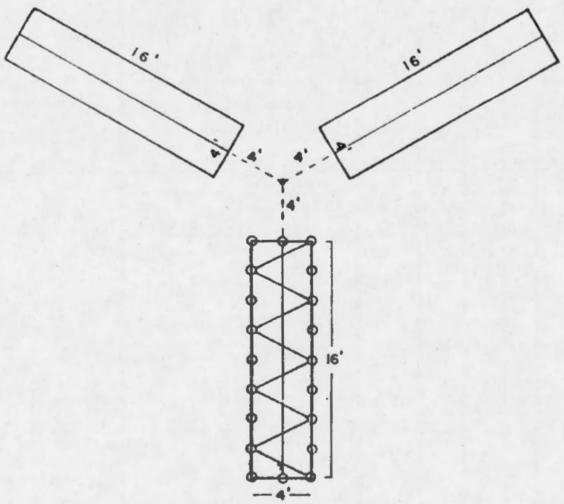
SAFE TARGET DIMENSIONS

HORIZONTAL CONTROL

Mapping Photography Scale 1/62,000

Identification Scale 1/25,000

Diameter = 40'



Materials

48' of 8 gauge, white, vinyl plastic

60 #1 Griff clips

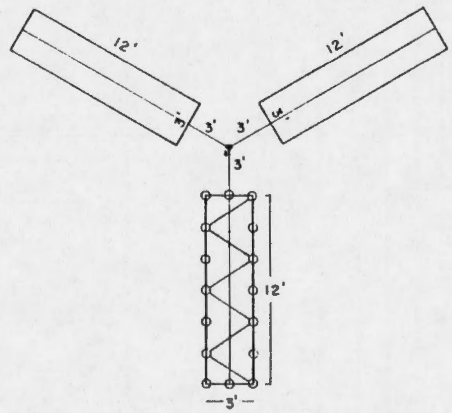
60 yds. Solid braided #3, Nylon Cord

60 8" or 12" Aluminum pegs

Mapping Photography Scale 1/50,000

Identification Scale 1/20,000 or 1/25,000

Diameter = 30'



Materials

36' of 8 gauge, white, vinyl plastic

48 #1 Griff clips

48 yds. Solid braided #3, Nylon Cord

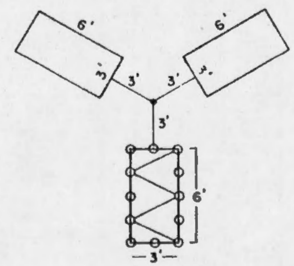
48 8" or 12" Aluminum pegs

VERTICAL CONTROL

Mapping Photography Scale 1/50,000 or 1/62,000

Identification Scale 1/20,000 or 1/25,000

Diameter = 18'



Materials

18' of 8 gauge, white, vinyl plastic

36 #1 Griff clips

36 yds. Solid braided #3, Nylon Cord

36 8" or 12" Aluminum pegs

APPENDIX B

SURVEY MARKERS

Markers used for supplementary control surveys include those recommended in "Specifications and Recommendations for Control Surveys and Survey Markers, 1978", (S & M, 1978) and the following types listed and supplied by Technical Field Support Services (TFSS).

The TFSS Field Equipment catalogue lists station markers and bench marks under "Bench Marks". Listed below are types of markers used, TFSS numbers, a brief description, and where they are used.

6675-21-876-1646: Survey tablet station markers (Type 1), horizontal control, bilingual, made of bronze, 79 mm in diameter, 70 mm shank and 17 mm slot for wedge at bottom. For use in bedrock, rock outcrops, large boulders, and concrete structures. Drill holes in uneven rock surfaces for this marker should be countersunk.

6675-21-876-1645: Survey tablet reference marker (Type 1), horizontal control, description and use same as above marker (1646).

6675-21-1644: Survey bolt station marker (Type 2), horizontal control, 25 mm in diameter and 89 mm in length with 13 mm wedge slot on bottom. To be used in place of horizontal control marker (1646) in areas where vandalism may be expected. This marker should always be referenced.

6675-21-3948: Survey tablet station marker, (Type 4), horizontal control, bilingual, cast aluminum, 76 mm in diameter. To be used with a steel reinforcing bar 1.2 m long (6675-21-876-3947), or 1 m long (6675-21-876-3946). A hole, about 15 mm in diameter, through the marker allows it to be forced over the bar. The marker is held secure by hammering and expanding the metal at the top of the rod. For use in low bearing soils and frozen ground. The rod has a tendency to become loose and unstable after being warmed in the sun in frozen ground or muskeg. It should not be used near towns because reinforcing steel rods are common and, if the marker tablet is removed, there is a chance of confusion.

6675-21-877-0387: Survey tablet reference marker, (Type 4), horizontal control, bilingual, same material and uses as above morker (3948).

6675-21-880-6978: Survey post station marker, horizontal control, bilingual, aluminum. The post is a 17 mm square bar, 1.2 m long with two spread bars 14 cm long at the bottom. A tablet 76 mm in diameter is attached by a tapered securing pin which can be adjusted to various heights using one of the four predrilled 6 mm holes at 15 cm spacing. The survey post is much lighter than the steel rod, an advantage when using helicopters. It has a distinctive shape, and the spreadfoot makes it difficult to remove.

6675-21-876-1677: Survey station marker, outside threads, bilingual, horizontal control, threaded in top of PVC pipe marker 6675-21-876-4216. The PVC marker consists of two PVC pipes, 4 1/2 and 6 5/8 inches diameter, that nestle for easy transport in a helicopter. The assembly has two reducers, and one blank flange 9 1/2 inches in diameter for the bottom plate. The hole for the marker is usally made by a 10-inch earth auger. This marker is used in light soils in remote areas where the cost of transporting heavier materials would be excessive. In use, the pipe is filled with the augered soil and the two pipes, flange and top, are melded together using PVC solvent. The pipes can be easily cut to fit the depth of the hole.

6675-21-876-1678: Survey tablet marker (Type 1), vertical control, bench mark, bilingual, made of bronze 79 mm in diameter, fits over copper clad ground rod (5975-21-876-2874) 2.4 m in length. These rods can be connected by means of a coupler (6675-21-876-3947) to give extra length. When driven in the ground with a jack hammer the rod is heated from the constant pounding and will penetrate permafrost. Also used in swampy ground where it is driven to refusal.

6675-21-876-3462: Survey tablet reference marker, rod type, horizontal control, description and use same as 3463.

6675-21-876-1651: Survey tablet bench mark, rod type, vertical control, description and use same as 3463.

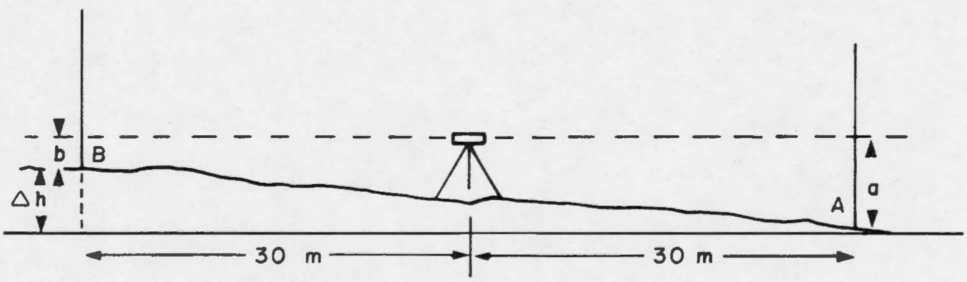
HELIX PIPE MARKER: Survey tablet bench mark, ideal for vertical and horizontal control, bronze 67 mm square, fits over 51 mm square standard heavy wall steel pipe of 2.44 metre length with a 45° bevel at the bottom. A 360° helix with a pitch of 9.5 cm is welded 15 cm from the bottom of the pipe. Pipe is normally screwed into ground using a specially adapted mobile drill rig, but may be installed manually by two people. Used in gravel, glacial till and fine grained soil. A majority of the inertial survey stations in Manitoba, Saskatchewan and Alberta are marked with this type of helix pipe marker using either square or round pipe.

APPENDIX C

TWO-PEG TEST FOR LEVELS

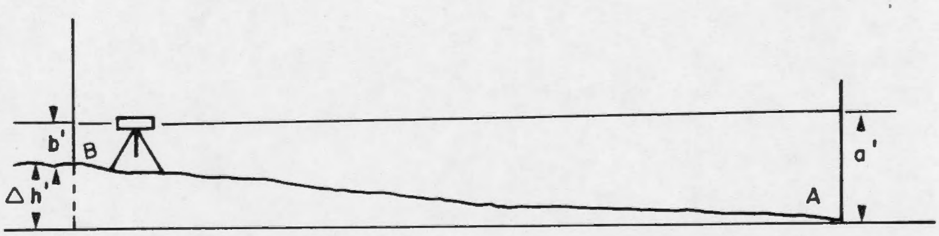
On firm and open ground select two points, A and B, 60 m apart with A slightly lower than B. Establish a firm and solid rod position at each point. Measure and mark the mid-point in line between A and B.

Set the instrument up at the mid-point and read carefully on each rod.



$\Delta h = a - b$

As the backsight on A and the foresight on B are equal in length the effects of any collimation error will be zero, and the best estimate of the true height difference between A and B will be obtained.



$\Delta h' = a' - b'$

Move the level close to rod B (3.5 m approx.) and again read on rod A and rod B.

If Δh and $\Delta h'$ agree within 2 mm, no collimation adjustment is necessary.

If Δh and $\Delta h'$ disagree by more than 2 mm the instrument should be adjusted to obtain agreement. After adjustment, the test should be repeated.

The method of collimation adjustment differs depending on the type of level being used; refer to the manufacturer's handbook for collimation adjustment procedure.

APPENDIX D

FIELD FILM LOG

	#1	#2		
CAM TYPE & SERIAL	HASSELBLAD 1234	RICOH 387574	DATE	85-08-31
MAGAZINE SER. NO.	400615	–	AREA	NFLD
LENS	1234	–	CPC	335317
ROLL #	11	9	OPER.	J.C. LAVERGNE
TYPE FILM	70 mm	35 mm	SUN ANGLE	45°
FILM ASA or EAFS	PLUS X 250	TRIX 400	CONDITIONS	SUNNY
F STOP	AUTOMATIC	AUTOMATIC	9 x 9 SCALE	1:50,000
SHUTTER SPEED	500	300		

STATION	LOCAL TIME	9 x 9 PHOTO	ROLL	#1 FRAMES	ROLL	#2 FRAMES	ALTIMETER	DIR.	TARGET
1018	15:00	A 26719 - 118	11	5-9	9	10	3,100	010	⊥
1019	15:05	A26719 - 99	11	10-13	9	11	3,000	040	⊥
1020	15:09	A26719 - 100	11	14-17	9	12	2,900	005	⊥
1021	15:16	A 26721 - 17	11	18-20	9	13-14	3,000	005	⊥
1022	15:21	A26721 - 52	11	21-24	9	15	3,300	030	⊥
1023	15:25	A26719 - 19	11	25-27	9	16	3,500	060	⊥
1024	15:30	A26719 - 26	11	28-30	9	17	4,000	020	⊥
1025	15:36	A26783 - 72	11	31-33	9	18	3,800	320	⊥
1026	15:41	A26722 - 85	11	34-36	9	19-20	3,700	005	⊥
1028	15:45	A26781 - 136	11	37-39	9	21	3,800	045	⊥
1029	15:49	A26781 - 118	11	40-42	9	22	4,000	000	⊥
1030	15:55	A26781 - 77	11	43-46	9	23	3,900	005	⊥
520121 WULFF	16:05	A26781 - 116	11	47-49	9	24	8,000	215	人
520159 RANGER	16:17	A26781 - 60	11	50-54	9	25	6,900	240	人
850003	16:29	A26781 - 56	11	55-57	9	26	6,700	159	人
790019	16:44	A26781 - 127	11	58-61	9	27	6,400	165	人

SCS 78-1 (REV. 86-04-09)

FIELD FILM LOG [SCS 78-1 (Rev. 86-04-09)]

1. HEADING INFORMATION

CAM. NO.: The serial number of the 70 mm camera. Note that both 35 and 70 mm cameras are normally used together with the 35 mm serving as back-up.

MAG. NO.: The magazine number for 70 mm camera.

LENS: Serial number of lens.

ROLL NO.: The roll number of film (assigned by party chief).

TYPE FILM: Tri X. Plus X, etc.

FILM ASA OR EAFS Setting on cameral and/or light meter.

f:STOP: Diaphragm opening (Hasselblad, Pentax and Ricoh have automatic control of the diaphragm opening).

SHUTTER SPEED: Camera setting (no adjustment on Vinten).

DATE Date photographs are taken.

AREA: The project area (i.e. Northern Ontario).

CPC: Project number assigned for Cost and Production Control System.

OPERATOR Name of person taking the photographs.

SUN ANGLE: Solar altitude above horizon (should be at least 20°).

CONDITIONS: Cloud cover.

2. RECORDING FORM

The form contains a number of columns for recording of data. This data includes

STATION: The number (or name if number is unknown) of the survey station being photographed.

LOCAL TIME: Time to nearest minute of identification photography.

9X9 PHOTO Number of mapping control photo on which station will be annotated.

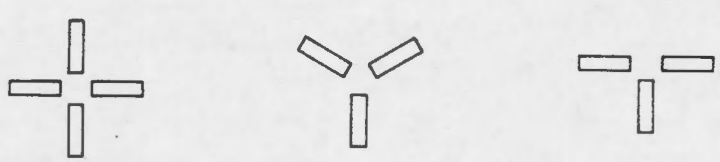
ROLL: The roll number for Camera #1 and Camera #2 film.

FRAMES: The number of exposures for each station.

ALTIMETER: The aircraft altimeter height above ground level assuming the altimeter has been recently set on a known elevation

DIR The aircraft heading at the times of exposure or the compass bearing or direction such as N.E. S., S.W., etc.

TARGET: A simple sketch of the type of target photographed should be shown here. This is in order to facilitate the transfer from ident-photography to mapping photography. Some common types are.



The length of the individual panel sizes should also be noted

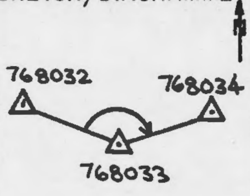
Page 1

ANGLES

DATE AUG. 1, 1976 INST./*APPAREIL* DKM2A # 206079 STA. 768033

OBS. DLP PHOTO A12107-167 ECCENTRIC/*ECCENTRIQUE* YES/*OUI* ☐ NO/*NON* ☒

STA.	HORIZONTAL READING/*LECTURE*	HORIZONTAL DIRECTION	MEAN/*MOYENNE*	TARGET/*MIRE*
768032	0-00-21			Light
"	180-00-25			"
	23	0-00-00		
768034	170-40-51			"
"	350-40-54			"
	52.5	170-40-29.5		
768032	202-30-23			"
"	22-30-19			"
	21			
768034	13-10-53			"
	193-10-48			"
	50.5	170-40-29.5		
768032	45-00-20			"
"	225-00-23			"
	21.5			
768034	215-40-48			"
"	35-40-50			"
	49	170-40-27.5		
768032	247-30-22			"
"	67-30-19			"
	20.5			
768034	58-10-51			"
"	238-10-48			"
	49.5	170-40-29.0		
TOTAL				
768032	SKETCH/*DIAGRAMME*	00° 00' 00"		
768034		170° 40' 27."3	(8 sets)	
			Spread 6"	



Checked by/*Vérifié par* R.C.

SCS 78-2

Page 2

ANGLES

DATE AUG. 1, 1976 INST./*APPAREIL* DKM2A #206079 STA. 768033

OBS. DLP PHOTO A12107-167 ECCENTRIC/*ECCENTRIQUE* YES/*OUI* ☐ NO/*NON* ☒

STA.	HORIZONTAL READING/*LECTURE*	HORIZONTAL DIRECTION	MEAN/*MOYENNE*	TARGET/*MIRE*
768034	270-00-25			Light
"	90-00-35			"
	30 ✓	0-00-00		
768032	99-20-03			"
"	279-20-05			"
	04 ✓	189-19-34.0 ✓	(170-40-26.0) ✓	
768034	112-30-35			"
"	292-30-33			"
	34 ✓			
768032	301-50-08			"
"	121-50-04			"
	06 ✓	189-19-32.0 ✓	(170-40-28.0) ✓	
768034	315-00-31			"
"	135-00-36			"
	33.5 ✓			
768032	144-20-07			"
	324-20-09			"
	08 ✓	189-19-34.5 ✓	(170-40-25.5) ✓	
768034	157-30-29			"
"	337-30-28			"
	28.5 ✓			
768032	346-50-07			"
"	166-50-03			"
	05 ✓	189-19-36.5 ✓	(170-40-23.5) ✓	
TOTAL				

SKETCH/*DIAGRAMME*

N

00° 00' 00"

Checked by/*Vérifié par* R.C.

SCS 78-2

HORIZONTAL ANGLES

1. **Heading information** - This information should be as complete as possible. The date, station number, and observer are of prime importance, as well as the serial number of the theodolite used. The photo number is required for identification purposes. A simple yes or no is required if there is any eccentricity involved in the measurement.

2. **Station Numbers** - The complete number must be used. Normally, numbers will be assigned by the Geodetic Survey Division and are of the form YYPNNN, where YY indicates the year of installation, P is a one-digit code representative of the province or territory, and NNN the unique station number. For example, station 716043 refers to station 043, in the province of Alberta, established 1971. Numbers from other agencies must be clearly identified. Stations involving eccentricities must be labelled as such.

   i.e. **728027 ECC** refers to the eccentric point

   while **728028** refers to the main station

3. **Readings** - One set consists of a direct and reverse reading on the initial, and direct and reverse readings on all other stations. The required order is:

   direct on initial (sta. 1)
   direct on sta. 2
   direct on sta. 3
   . . .
   . . .
   . . .

   direct on sta. n

   reverse on sta. n
   reverse on sta. n - 1
   . . .
   . . .
   . . .

   reverse on sta. 3
   " " sta. 2
   " " sta. 1

   For the second-order accuracy a minimum of six sets of horizontal angles, evenly distributed over the horizontal circle, must be observed (S & M, 1978).

The initial pointing for the first set on angle measurements is set near 0°00'00", and the circle reading for the initial pointing of each consecutive st of angle measurements is advanced by 30°.

In the example (page D2-1), to ensure that second-order accuracy will be achieved, eight sets of horizontal angles, evenly distributed over the horizontal circle, were observed. As in the six-set measurement, the initial pointing is set near 0°00'00" and the circle reading for the initial pointing of each consecutive set of angle measurments is advanced by 22°30'.

To eliminate a minute accumulative error, mainly attributed to strain of the theodolite, a small refinement in measuring technique can be introduced.

e.g.:At the three consecutive intervisible stations A, B, C, the targets are set up on stations A & C and the theodolite on station B. "A" is used as backsight and "C" is used as foresight, and the angle observed is angle ABC. After completion of the first four sets of observations, the theodolite is repointed on "C" (which will now be used as backsight) the horizontal circle set on 90°0'00", and the complementary angle CBA is measured for the next four sets.

The same technique can be applied to six sets of angle measurements; however, the complementary angle is measured after three initial sets.

By subtracting the complementary angle from 360°, we obtain values for angle ABC.

If the eight sets of horizontal angles have a spread exceeding 10", four additional sets must be measured.

4. **Direction - Mean** - Directions consist of a direct and reverse for each station. The directions on the initial are made equal to zero and the readings on each other station reduced by the readings on the initial to derive the directions at these stations.

The mean is simply the mean of the two directions. For example,

STATION	READING	DIRECTION	MEAN
1	D_1	00°00'00"	00°00'00"
	R_1	00°00'00"	
2	D_2	$D_2 - D_1$	$\frac{(D_2-D_1+R_2-R_1)}{2}$
	R_2	$R_2 - R_1$	

In computing directions, it is important to subtract directions in each set completely. The practice of obtaining the degrees and minutes from the first set and then only computing the seconds portion of each additional set is dangerous if a mistake is made in the first set. All sets should agree in degrees and minutes as well as seconds.

5. **Target** - This column is used to record the type of target being sighted. Examples include mirror flash, sealed beam, signal cloth, top of tower, range pole, etc.

6. **Sketch** - The sketch is an important part of the notes, particularly if the station numbers are later reversed. Except for angles near 180°, the sketch would quickly show whether the derived angle is from A to B or from B to A. Where more complex sketches are necessary, they should be put on the back of the recording form and the note "see back" entered in the block marked sketch.

7. **Notes** - Where convenient, the observer should make notes regarding the observations. These should include comments on weather, wind velocity, stability of set-up, visibility, steadiness of lights, etc.

Page 1

Tellurometer Distance Record

DATE July 6, 1972

AREA Keewatin Dist NWT

TELLY TYPE MRA3

REMARKS Clear, Cool

	MASTER	REMOTE
STATION	728027	728028
OPERATOR	RM	JW
SERIAL NO	14913	15297
DIPOLE HEIGHT	1.52 ~~ft~~/m	1.46 ~~ft~~/m
ALTIMETER NO	1538	D67-21

B	1	2		
C	2	3		
D	3	0		
E	1	6		
A	6	0	4	

B	1	2		
C	2	4		
D	3	3		
E	1	7		
A	6	6	7	

Uncorrected Distance

1 2 3 1 6.6 2 ~~ft~~ m

CAVITY	READING	MEAN	READING	CAVITY
40	664 / 667	666 / 672	674 / 670	130
50	660 / 651	656 / 675	677 / 673	140
60	657 / 658	658 / 678	679 / 678	150
70	642 / 645	644 / 680	679 / 681	160
80	639 / 642	640		
90	645 / 650	648		
100	653 / 659	656		
110	663 / 667	665		
120	669 / 668	668		
	MEAN	662		

METEOROLOGICAL READINGS – THIS STATION

	TIME	Td°C	Tw°C	ALTIMETER	CORR	CORR ALT	N	MEAN N
Remote to page 2	14:12	17.7	13.8	1953	-1008	945		
Master	14:19	17.3	13.4	1957	-1008	949		
	14:30	17.2	13.3	1958	-1008	950		
mean		17.3	13.4			950		

METEOROLOGICAL READINGS – OTHER STATION

	Td°C	Tw°C	CORR ALT	N
mean	14.0	12.2	1338	

Checked by / Vérifié par R.C.

SCS 78-3

TELLUROMETER DISTANCE RECORD

1. Heading Information - The date, station numbers (both master and remote) and observer are of prime importance and must be recorded. Where eccentric stations are used, it is necessary to indicate this on the station number (i.e. 728028 ECC). The tellurometer type is required, first, to verify the units of measurement (feet or metres) and secondly, in cases where zero or tilt errors may be applicable. The serial numbers allow a continuing monitoring of the performance of each instrument and also allow for frequency corrections to be added at a later date.

   The dipole heights are used in sea-level reduction of the observed, met-corrected distance. Where both are simply tripod heights, then neglecting this information will not introduce significant errors. However, in the case where one end of the line involves a large tower, then the heights are essential. It is good practice to always record these heights.

   The altimeter serial numbers must be noted so that calibration corrections may be applied. The pressure observations are used in meteorological corrections. The corrected readings may also serve as a gross check on the observed elevation difference (by trig heighting).

2. Coarse Distance - At least two coarse distances are required at each end. Checks for ambiguous data must be made at the time of observation, and the reduced coarse distance confirmed by the remote operator.

3. Fine Readings - The fine readings on each selected frequency (or cavity) is recorded and a mean computed. The frequency or cavity reading should be noted. This is useful where the distance is distorted by secondary reflections along the line. This ground swing is noted by the spread of fine readings. Normally, the difference between the highest and lowest mean fine reading will be less than 0.5 metre. Where larger differences occur, the cavity reading and corresponding mean fine reading can be plotted to study the ground swing. If it is systematic, appropriate corrections can be applied. Very large differences (1.0 metre) should not normally be accepted. Steps should be taken to reduce the problem of secondary reflections (tilting the tellurometer, raising the instrument height, using ana eccentric, etc.).

   Where it is not practical to eliminate large swings (i.e. long lines over water or ice), then the only alternative is to ensure there are sufficient observations. The coarse distance in particular must be confirmed.

4. Uncorrected Distance - The mean fine reading and the reduced coarse distance are combined to form the uncorrected slope distance. The units must be shown as we have tellurometers which measure in feet and in metres.

5. Meteorological Readings - Normally, three sets of met readings are taken at each station during measurement. They are:

   1. Start of master measurement at station A.
   2. End of master measurement at station A. This set is also used as the start of master measurement at station B.
   3. End of master measurement at station B.

   The times should be recorded so that the met readings may be correctly grouped.

   The meteorological correction is derived from the mean mets on each end of the line.

   There will be two distances measured for the line AB, one measured from A and the other from B. Two met corrections will be computed for each distance. To the distance measured at A:

   - met correction using the mean of data observed before and after at station A
   - met correction using the mean of data observed at station B at the same times

   Similarly there will be two met corrections for the distance measured at B.

   The mean met correction at each end is computed and the distance from each end corrected accordingly. The final slope distance is then the mean of the two corrected distances.

   The recording form is set up so that the mean met at that station can be recorded as well as the corresponding mean from the remote station.

   Altimeter calibration corrections are noted in the column labelled "CORR".

   Where nomograms are used, the two values of the refractive index are computed instead of met corrections. The mean index is then used to correct the distance.

6. Example - The attached example is a measurement of the line 728028 to 728027. The form should be self-explanatory.

Page 1

VERTICAL ANGLES / *ANGLES VERTICAUX*

Instrument/*Appareil* DKM2A# 206079 DATE July 28, 1976 STA 758001

OBS. DLP   T1 1·85 metres O1 1·63 metres O2 1·28 metres

TO STA. *VERS STATION*	TIME *TEMPS*	READING *LECTURE*	ZENITH DISTANCE *LA DISTANCE ZÉNITHALE*	MEAN *MOYENNE*
768018	10:00	88-15-25	88-15-25✓	88-15-23.0✓
		271-44-39	88-15-21✓	
		360-00-04		
768018	10:04	88-15-20	88-15-20✓	88-15-18.0✓
		271-44-44	88-15-16✓	
		360-00-04		
768018	10:06	88-15-23	88-15-23✓	88-15-21.0✓
		271-44-41	88-15-19✓	
		360-00-04		
768018	10:08	88-15-24	88-15-24✓	88-15-20.5✓
		271-44-43	88-15-17✓	
		360-00-07		
		MEAN / *MOYENNE*		88-15-20.6✓

REMARKS / *REMARQUES* Spread 5"

Sunny & clear, light wind.

Target: sealed beam

T1 Instrument height this station / *Hauteur du appareil cette station*
O1 Target height this station / *Hauteur de la mire cette station*
O2 Target height other station / *Hauteur de la mire autre station*

Checked by *Vérifié par* R. C.

SCS 78-5

ZENITH DISTANCES (Vertical Angle Form SCS 78-5)

1. Heading Information - This information must be complete. The instrument type and number is important since the reduction of observations may be different for different instruments (i.e. T-2 observtions as opposed to T-3 data). The date is required, particularly if remeasurements of the line are observed. After the field season, the dates usually provide the only means of matching the correct sets of observations. Zenith distances measured simultaneously from both ends of the line are needed to compute the elevation difference accurately.

   The largest source of problems with trigonometric height computations has always been missing or incorrect instrument and signal heights. An error in heights causes an equal error in the computed elevation difference. The symbols T for instrument height and 0 for target height have been adopted. The observing station is given the subscript 1 (T_1, 0_1) and the observed station the subscript 2 (0_2) - regardless of actual station numbers. For example, a zenith distance measured at 758001 to a target at station 768018 would show T_1 and 0_1 as the instrument and target heights, respectively, at 758001. 0_2 would refer to the target height at 768018. The value of 0_2 may have to be entered back at the field camp, since communication problems on the line may make it impractical to pass this information during the observations.

2. "To Station" - The observed station number is entered here. Only one observed station should go on any one page. Where two or more stations are recorder on the same page, care must be taken to ensure the target heights are clearly explained. Note that observations to two stations may involve two values of 0_1, and certainly two values of 0_2. The notes must show which height refers to each station.

3. Time - The times of observation are necessary to show that observations are simultaneous. They become particularly important if 2 or more measurements are made on the same line on the same day. This may occur if observing conditions are not acceptable on the first measurement and it is decided to repeat the measurement later in the day. The times will provide the only means of correctly matching the observations.

4. Reading - Normally, a measurement consists of four sets, with a set consisting of both direct and reverse zenith distance observations. While the form does not provide space, the experienced observed will add the direct and reverse readings. The difference between this sum and 360° provides a measure of the vertical collimation error. A large change in this error may indicate problems with the set such as pointing errors or dislevelment.

5. Zenith Distance - The reduced zenith distance is usually equal to the direct reading on most "1" instruments (T2, DKM2A, etc.) and given by 360° - reverse reading. The mean is the arithmetic of the direct and reverse reduced zenith distances. The method of reduction will depend on the instrument used. For example, with Wild T3 observations, the zenith distance is given by:

   $$Z.D. = (D - R) + 90°$$

   where D = direct reading

   R = reverse reading

6. Other Notes - The range or spread is usually noted. This range should be less than 8 arc-seconds for four sets. Where higher ranges occur, or where the observer suspects the observations are of inferior quality, additional sets are measured.

   The remarks should show weather conditions and the type and stability of target. The type of target sighted is important. For example, if a tower is sighted, the notes must show where on the tower pointings were made so that a correct target height may be assigned.

D4-3

Page 2

VERTICAL ANGLES / *ANGLES VERTICAUX*

Instrument/*Appareil* DKM3A # 169853 DATE JULY 28, 1976 STA 768018

OBS. W.R. T_1 1·45 metres O_1 1·28 metres O_2 1·63 metres

TO STA. *VERS STATION*	TIME *TEMPS*	READING *LECTURE*	ZENITH DISTANCE *LA DISTANCE ZÉNITHALE*	MEAN *MOYENNE*
758001	10:00	91-59-46	91-59-46	91-59-45·5
		268-00-15	91-59-45	
		360-00-01		
758001	10:04	268-00-19	91-59-41	91-59-45·5
		91-59-50	91-59-50	
		360-00-09		
758001	10:06	91-59-50	91-59-50	91-59-47.0
		268-00-16	91-59-44	
		360-00-06		
758001	10:08	268-00-18	91-59-42	91-59-45·5
		91-59-49	91-59-49	
		360-00-07		
		MEAN / *MOYENNE*		91-59-45·9

REMARKS / *REMARQUES* Spread 1·5"

Sunny & clear, light wind.

Target: sealed beam

T_1 Instrument height this station / *Hauteur du appareil cette station*

O_1 Target height this station / *Hauteur de la mire cette station*

O_2 Target height other station / *Hauteur de la mire autre station*

Checked by *Vérifié par* R.C.

SCS 78-5

LEVELLING

Page 1

DATE July 29, 1972 OBS. J.W. ⊼ D412 A64

FROM BM / DU REPÈRE S-64 TO / AU 728028

UNITS : FEET

BACKSIGHT-*VISÉE ARRIÈRE*			FORESIGHT-*VISÉE AVANT*		
READING *LECTURE*	MEAN *MOYENNE*	INTERVAL *INTERVALLE*	READING *LECTURE*	MEAN *MOYENNE*	INTERVAL *INTERVALLE*
5.621			6.705		
3.871	3.871 ✓	1.750 ✓	4.955	4.955 ✓	1.750 ✓
2.121		1.750 ✓	3.205		1.750 ✓
4.027			7.642		
2.407	2.407 ✓	1.620 ✓	5.742	5.742 ✓	1.900 ✓
0.787		1.620 ✓	3.842		1.900 ✓
6.037			5.398		
4.037	4.037 ✓	2.000 ✓	3.648	3.648 ✓	1.750 ✓
2.037		2.000 ✓	1.898		1.750 ✓
4.697			7.086		
3.197	3.197 ✓	1.500 ✓	5.166	5.166 ✓	1.920 ✓
1.697		1.500 ✓	3.246		1.920 ✓
4.657			6.938		
2.557	2.557 ✓	2.100 ✓	5.036	5.036 ✓	1.902 ✓
0.457		2.100	3.134		1.902 ✓
3.972			5.387		
2.472	2.472 ✓	1.500	3.887	3.887 ✓	1.500 ✓
0.972		1.500	2.387		1.500 ✓
7.631			5.400		
4.921	4.921 ✓	2.710	2.941	2.941 ✓	2.459 ✓
2.211		2.710	0.482		2.459 ✓
SUM/*SOMME*	23.462 ✓	26.360 ✓	SUM/*SOMME*	31.375 ✓	26.362 ✓

F.S./V.A.	-31.375 ✓
DIFF.	-7.913 ✓
ELEV. S-64	1042.820 ✓
ELEV. 728028	1034.907 ✓

Checked by / *Vérifié par* R.C.

D5-2

LEVELLING

Page 2

DATE AUG. 4, 1976 OBS. J.W. ⊼ D412 A64

FROM BM / *DU REPÈRE* 59A121 TO / *AU* 728043

UNITS : FEET

BACKSIGHT–*VISÉE ARRIÈRE*			FORESIGHT–*VISÉE AVANT*		
READING *LECTURE*	MEAN *MOYENNE*	INTERVAL *INTERVALLE*	READING *LECTURE*	MEAN *MOYENNE*	INTERVAL *INTERVALLE*
4.126			8.337		
2.636	2.636✓	1.490✓	6.817	6.817✓	1.520✓
1.146		1.490✓	5.297		1.520✓
3.802			7.304		
2.182	2.185✓	1.620✓	5.684	5.684✓	1.620✓
0.572		1.610✓	4.064		1.620✓
4.124			9.494		
2.374	2.374✓	1.750✓	7.764	7.762✓	1.730✓
0.624		1.750✓	6.029		1.735✓
Intermediate sight to			7.385	*	
C.H.S. stn. D-28			5.985	(5.985)✓	1.400✓
From same set up as			4.585		1.400✓
previous foresight					
4.667			10.047		
2.993	2.996✓	1.674✓	8.377	8.377✓	1.670✓
1.327		1.666✓	6.707		1.670✓
4.515			5.202		
2.715	2.715✓	1.800✓	3.352	3.351✓	1.850✓
0.915		1.800✓	1.499		1.853✓
SUM/*SOMME*	12.906✓	16.650	SUM/*SOMME*	31.991✓	16.788
F.S./V.A.	− 31.991✓				
DIFF.	− 19.085✓				

* Not included in sum.

Checked by / *Vérifié par* R.C.

SCS 78-6

Elev. Diff. 59A121 to CHS D-28 = −11.291✓

SPIRIT LEVELS (SCS 78-6)

1. Heading Information - This information includes the date, observer and instrument serial number. The latter is required in case a particular instrument is later found to be defective or seriously out of adjustment. All observations with this instrument can then be carefully checked.

   The form is designed for differential levelling where the elevation of the end point is the only requirement, as opposed to profile levelling where the elevations of points along the line are required. Consequently, the form allows for recording of only starting and ending point names.

   Where the line is sufficiently long so that all the information cannot be put on one page, then the line is broken into sections with one page per section. For example, the line from BM A to Station 22 might require three sections. The three pages would then be labelled.

   From BM: BM A to TBM 1
   From BM: TBM 1 to TBM 2
   From BM: TBM 2 to Sta 22

   The TBMs (temporary bench marks) must be stable points chosen such that they can be recovered, at least until the levelling is completed. One reason for sections is to minimize the amount of relevelling necessary when misclosures are unacceptably high. All levelling must be two-way, that is, levelled from BM to station then from station to BM. The difference between the two elevation differences must be within allowable limits (see Section 4.1.5, Accuracies). The forward minus back differences for each section must be noted. Where gross errors have occurred, the particular section will be relevelled instead of the entire line.

2. Three-Wire Levelling - The form is designed for three-wire levelling. That is, each sighting consists of three rod readings corresponding to the three horizontal cross hairs. The mean reading (mean of all three) must be reasonably close to the centre reading.

   The "interval" is used to compute the stadia distance. The "interval" is the difference between the top and lower rod readings. The distance in feet, instrument to rod, is obtained by multiplying the "interval" by the instrument constant (usually 100).

   This stadia distance is noted and the sum of backsight and foresight stadia are compared. The requirement is that backsights and foresights be balanced. Accumulative totals can be used and a balance noted at each set-up. The next backsight (or foresight) can then be correspondingly lengthened or shortened to maintain equal backsight and foresight distances.

3. Example (labelled "Page 1") - The example shows levelling data between bench mark S-64 and station 728028. Note that only the forward levelling for this line is shown.

   The stadia interval has been determined by subtracting the centre reading from the top and the bottom readings - resulting in two numbers. If these two agree, then the centre reading is correct and is copied into the "mean" column. The actual stadia distance is the sum of the two interval numbers.

   The allowable difference between the two intervals is governed by the type of levelling and the required final accuracy. Where maximum precision is desired, the two stadia intervals should not be greater than 0.01 foot. Larger differences require reobserving.

   The sums of the mean backsights and mean foresights are recorded. The elevation difference then becomes: Σbacksights - Σforesights and is noted as "DIFF" on the form.

   Whenever problems are encountered, or where abnormal situations occur, clear notes should be made on the form. If necessary, explanations should be made on the back of the form and the note "see back" put on the front.

4. Example 2 (labelled "Page 2") - The difference between example 2 and 1 is the use of an intermediate sight (I.S.). This is a common occurrence and usually involves establishing the elevation of some point along the line, but a point which is not part of the line. Example 2 shows a vertical tie to Hydrographic station D-28. Note that it is very important to show how this sight was made. The example indicates the I.S. was taken on the same set-up as the preceeding foresight. Therefore, in computing an elevation difference from the bench mark to D-28, this preceeding foresight will not be included.

   Care must be taken to ensure the I.S. is not included in the sum of foresights and the sum of foresight intervals.

RECIPROCAL VERTICAL ANGLE

TRAVERSE STARTING AT STATION # PM 04 ELEVATION ______ AREA ______

TRAVERSE ENDING AT STATION # 809224 ELEVATION ______ MAP SHEETS ______

STATION	STATION	DISTANCE (m)	DRY °c	WET °c	Hg "	SLOPE DISTANCE	O_1/t_1	O_2/t_2	ZENITH DIST. 1 / ZENITH DIST. 2	ΔH	ELEVATION	SEA LEVEL DIST.	HORIZONTAL ∠
1 PM04	2 PM 04A	586.213	17.2	11.9	29.25	586.222	1.56 / 1.56	1.41 / 1.41	94 - 19 - 20.8	-44.008			
2 PM 04A	1 PM 04	586.216	17.5	11.9	29.25	586.225				-44.008✓			
							1.41 / 1.41	1.56 / 1.56	85 - 40 - 56.8	44.008			
1 HOW A	2 HOW	1896.807	18.3	12.5	29.20	1896.837	1.70 / 1.70	1.62 / 1.62	88 - 37 - 56.5	45.818			
2 HOW	1 HOW A	1896.755	18.5	12.5	29.20	1896.786				45.818✓			
							1.62 / 1.62	1.70 / 1.70	91 - 23 - 44.7	-45.817			
1 809219A	2 809219	2022.391	23.8	18.0	29.13	2022.335	.75 / 1.60	1.29 / 1.51	88 - 53 - 43.8	+39.487			
2	1												
									91 - 08 - 40.8				
1 809219A	2 809219	2022.412	20.2	12.6	29.08	2022.460	1.77 / 1.77	1.52 / 1.52	88 - 53 - 55.4	39.414			
2 809219	1 809219A	2022.436	20.3	12.7	29.08	2022.483				-39.414✓			
							1.52 / 1.52	1.77 / 1.77	91 - 07 - 04.3	-39.414			
1 809219	2 809219A	2022.449	20.1	12.9	29.08	2022.492	1.52 / 1.52	1.77 / 1.77	91 - 07 - 04.3	-39.414			
2 809219A	1 809219	2022.403	20.1	12.9	29.08	2022.446				-39.414✓	MEAN 39.414		
							1.77 / 1.77	1.52 / 1.52	88 - 53 - 55.4	+39.414			
1 809224A	2 809224	1608.439	18.0	12.3	29.32	1608.463	1.62 / 1.62	1.69 / 1.69	88 - 42 - 57.4	+36.360			
2 809224	1 809224A	1608.436	18.2	12.5	29.31	1608.459				-36.360✓			
							1.69 / 1.69	1.62 / 1.62	91 - 18 - 41.7	-36.360			
1	2												
2	1												
1	2												
2	1												
1	2												
2	1												
1	2												
2	1												

86-04-28

RECIPROCAL VERTICAL ANGLE ABSTRACT

This form may be used as an abstract of the information from the field notes, or as a record-keeping form for field processing. Most of the information it contains is self-explanatory.

COLUMNS 1 & 2 - The "FROM" and "TO" station numbers or names respectively.

COLUMN 3 - The measured slope distance at each station.

COLUMNS 4 & 5 & 6 - Dry and wet bulb temperatures measured to the nearest .1°C and pressure measured in inches of mercury. These values represent the mean of the met readings at the station taken before and after the distance measurements.

COLUMN 7 - The slope distance corrected for met observations.

COLUMN 8 - The theodolite (t_1) and target (0_1) heights measured to the nearest centimetre at the "FROM" station.

COLUMN 9 - The theodolite (t_2) and target (0_2) heights measured to the nearest centimetre at the "TO" station. Note that the 0_1 and t_1 values at the "FROM" station should be the same as the 0_2 and t_2 values at the "TO" station and vice versa.

COLUMN 10 - The mean of the Zenith Distances from the vertical angle field notes for stations 1 and 2 respectively.

COLUMN 11 - The computed height differences from stations 1 and 2 respectively. The value in the box represents the mean ΔH between these two stations with proper consideration for differences in sign.

COLUMN 12 - The elevation for each point is normally carried forward in a vertical angle traverse computation. The elevation at station 2 represents the elevation at station 1 plus ΔH from column 11.

ALTIMETER TRAVERSE FIELD COMPUTATIONS (FORM SCS85-IR)

PROJECT:	Cost and Production Control (CPC) project number.
LOCATION:	Project Area.
OPERATOR:	Initials of Observer.
HELICOPTER:	Type and call sign of helicopter used.
LINE:	Number assigned by party chief.
DATE:	Year, month and day of observarions.
ALT. #	Serial numbers of 3 altimeters used in observations.
STATION #:	National Geodetic Data Bank (NGDB) unique number in case of existing station. For non-marked stations, number assigned by part chief. Normally line number followed by sequential numbers.
PHOTO:	9 x 9 photo number on which station appears.
S ON STATION OR H in the HELICOPTER:	Enter S if altimeters are read at ground on the station mark. Enter H if altimeters are read in the helicopter.
H TO STATION:	Correction value to be applied to readings taken at ground level outside helicopter. This value is determined for each individual helicopter.See "Helicopter/ Ground Corrections", Section 4.3.6.
TIME:	Time of altimeter readings.
ALITIMETER 1, 2, 3:	Readings of the altimeters.
CORRECTION (CAL)	Calibration correction value at the individual altimeter readings.
VALUE 1,2,3:	Altimeter readings minus calibration corrections.

SPREAD: Largest difference between any two of the three
altimeter values.

MEAN: Mean of the three values.

TEMPERATURE: Temperature in degrees read 40 metres above station.

DRY °F/°C: Circle °F or °C as applicable.

HUMIDITY % OR WET °F/°C Humidity or Wet Bulb reading taken 40 metres above station. Note which is used by circling.

ACCUMULATED DISTANCE KM: Distance in kilometres to each station at zero on the first control station. Used to adjust residual error.

LAWNIKANIS GRID: Location of the station on the 9 x 9 photo to the nearest millimetre, obtained using the "Lawnikanis" template. See Section 4.3.7, "Recording".

NOTES: Used to note station eccentricities, known elevations, wind speed and direction and other pertinent information.

ALTIMETER TRAVERSE FIELD COMPUTATIONS

PROJECT: 335320

LOCATION: N.B.

OPERATOR: J.W.

HELICOPTER: Hughes 500D CF-BCW

LINE: 11 EAST

DATE: 85/10/8   ALT.# 10863   ALT.# D70-8   ALT.# 67-16

STATION #	PHOTO	S on station / H in helicopter	h to station	TIME	ALTIMETER NUMBER and	CORRECTION (CAL)	VALUE 1	ALTIMETER NUMBER and	CORRECTION (CAL)	VALUE 2	ALTIMETER NUMBER and	CORRECTION (CAL)	VALUE 3	SPREAD	MEAN	TEMPERATURE DRY °F	HUMIDITY OR WET °F °C	ACCUMULATED DISTANCE KM	LAWNIKANIS GRID	NOTES
1118G-X		H	0	13:19	655			665			661									W 10 mph
1118G-X ✓	A26694-44	H	0	13:20	655	29	684 ✓	665	19	684 ✓	661	21	682 ✓	2 ✓	683 ✓	51.8	32.0	0	R.2×15.2	1118G ECC ELEV= 31.27
1127 ✓	26694-40	H	0	13:35	990	27	1017 ✓	995	23	1018 ✓	996	20	1016 ✓	2 ✓	1017 ✓	52.1	33.0	27	I.0×6.5	W 10
1027 ✓	26694-37	H	0	13:41	672	29	701 ✓	684	20	704 ✓	680	21	701 ✓	3 ✓	702 ✓	52.7	30.4	40	I.2×20.8	W 10
1026 ✓	26691-100	H	0	13:46	1150	26	1176 ✓	1155	23	1178 ✓	1157	18	1175 ✓	3 ✓	1176 ✓	51.4	31.0	56	P.1×20.5	W 10
1111 ✓	26694-33	H	0	13:51	1556	24	1580 ✓	1560	23	1583 ✓	1568	13	1581 ✓	3 ✓	1581 ✓	51.6	31.9	65	N.6×20.8	W 10
1025 ✓	26691-94	H	0	13:57	1890	25	1915 ✓	1895	18	1913 ✓	1900	13	1913 ✓	2 ✓	1914 ✓	51.7	30.8	83	V.7×5.5	W 10
1024 ✓	26694-26	H	0	14:08	1921	25	1946 ✓	1930	17	1947 ✓	1932	13	1945 ✓	2 ✓	1946 ✓	48.5	35.2	109	J.9×20.5	W 10
801564 ✓	—	H	0	14:28	1900	25	1925 ✓	1906	17	1923 ✓	1910	13	1923 ✓	2 ✓	1924 ✓	49.2	37.5	144	—	801564 ELEV= 419.28 m
801564		H	0	14:29	1900			1906			1910									W 10

D7-3

Checked by: G.M.

INERTIAL SURVEY RECORD

R - Roll
F - Frame

DATE: 86/07/22 ALIGNMENT: 203a OPERATOR: M.G. RUN: #4 W

TRK RNG	STATION	LATITUDE	LONGITUDE	ELEVATION	OFFSETS BRG	OFFSETS DIS	OFFSETS DEL	AO TIME	C1 PNT	PHOTOS R	PHOTOS F	TARG COND	STAMPED NUMBER	MKR CODE	COMMENTS
0	Ecc 779443	45-59-28.621	71-53-45.346		25	.18	-105	1-11-45	10	1	10	OK	779443	K	UPDT. - H. PIPE ON ROAD
1	769442	45-58-59.312	71-55-32.917	149.82	40	.28	-119	1-18-21	11/12	1	13	POOR	769442	D	BRASS PLUG H.&V. UPDT.
2	76~~8~~9441	45-59-30.	71-58-10		28	.11	+10	1-23-50	13	1	18	OK	769441	D	BRASS PLUG HIGH WINDS
3	769440	45-57-25	71-58-25		35	.15	-125	1-30-10	14	1	21	OK	NO STAMP	D	BRASS PLUG
4	769439	45-58-31.978	71-59-52.9		05	.19	-101	1-35-12	15	1	24	NONE	769439	D	HARD TO FIND STATION BRASS PLUG HORZ. UPDT.
5	6410	45-59-12	72-00-10	197.35	02	.34	-106	1-41-39	16	2	1	NONE	6410	F	BM. PIPE VERT. UPDT.
6	769220	45-59-15	71-59-05		74	.48	-115	1-45-10	17	2	5	POOR	769220	A	AL. CAP
7	769221	45-59-05.724	71-57-31.912	201.72	13	.22	-110	1-51-47	18/19	2	10	OK	769221	A	AL. CAP H.&V. UPDT.
1	2	3			4			5	6	7		8	9	10	11

D8-1

INERTIAL SURVEY RECORD

1. Track and Range (TRK, RNG) - In this column the identification number is entered. This function on the CDU enables the operator and pilot to navigate the vehicle to proposed station sites. Refer to ISS operators manual for procedures to enter track and range co-ordinates.

2. Station - Enter station NGDB unique number. (This number may not be the same as the number stamped on the monument.)

3. Latitude, Longitude and Elevation - Enter map scaled or known co-ordinates.

4. Offsets (BRG, DIS, DEL) - **Bearing** : Enter the angle measured on the offset protractor, in degrees. **Distance:** Enter the distance in centimetres, measured from the offset protractor. **Delta Elevation:** Enter the difference in elevation, in centimetres, from the offset protractor. This elevation difference is from the monument to the offset device.

5. AO Time - This is the location on the CDU thumb wheel that indicates the time the operator entered a mark and/or udate. The TIME is recorded in this column.

6. CI PNT. - In this location, the point number of a mark and/or update is entered.

7. Photos - Roll or Frame - In this column, 35 mm picture roll number and frame numbers are entered of the photos taken of the marker.

8. Target Condition - In this column, the operator indicates the shape of the target, and the condition.

9. Stamped Number - The actual number and/or letters observed by the operator that are stamped on the monuments are recorded in this column.

10. Marker code - Refer to Geodetic Survey of Canada Form #58 for the MARKER CODE (i.e. A - aluminum tablet).

11. Comments - Indicate any variances from the planned survey mission.

OPERATOR ENTRY ERRORS

			ENTERED VALUES				CORRECT VALUES				
DATE	ALIGN I.D.	PNT	STATION	OFFSETS BRG°	OFFSETS DIS(cm)	OFFSETS DEL(cm)	STATION	OFFSETS BRG°	OFFSETS DIS(cm)	OFFSETS DEL(cm)	COMMENTS
86/07/22	203a	2	768441				769441				incorrect station#
86/07/22	203a	3				+125				-125	incorrect sign of del. elevation
86/07/22	203a	7		000°				13°			incorrect bearing entered
1	2	3	4	5	6	7	8	9	10	11	12

OPERATOR ENTRY ERRORS (ISS)

1. Date - Enter year, month and day.

2. Alignment I.D. (ALIGN ID) - Enter the Julian date followed by a letter representing the alignment number:

   i.e. Letter "a" is the first alignment
   Letter "b" is the second alignment, etc.

3. Point number (PNT) - Enter the point number of the mark or update from column 6 of the Inertial Survey Record or from the computer print-out.

   Entered Values

4. Station - Enter the station number from the computer processing list only if the number differs from the correct station number.

5. Bearing (BRG°) - Entered in degrees only.

6. Distance (DIS) - Enter in centimetres only.

7. Delta Elevation (DEL) - Enter in centimetres only.

8.
9.
10.
11. Correct Values - Enter the correct value from the Inertial Survey Record Form in these columns. At this time, correct the field processing disk.

12. Comments - Enter any comments pertaining to the corrections.

VERTICAL SMOOTH DISC # 1

CPC # 335309

FILE	DATE	# PNTS	STATION	RUN #	RUN LENGTH	OPR	PRO
JUL31a6	85-07-31	8	699030 to 85T2034	10 S	52 km	CT	CC ✓
JUL31a7	85-07-31	7	85T2034 to 699037	10 S	45	CT	CC ✓
JUL31a9	85-07-31	9	BCH-4 to 85T2080	9S	57	CT	CC ✓
JUL31a12	85-07-31	9	85T2080 to BCH-4	9N	57	CT	CC ✓

VERTICAL SMOOTH DISC

FILE - Enter: 1) the month - 3 letters only
2) the day
3) the letter representing the alignment identification
4) the disc file number

DATE - Enter the year, month and day

PNTS - Enter the number of points (marks and vertical updates) for the vertical traverse.

STATIONS - Enter the initial and final vertical update station of the traverse.

RUN # - Enter the run (traverse) number and the direction of travel: (N - north ; S - south , E - east ; W - west).

RUN LENGTH - Enter, in kilometres, the distance of the traverse.

OPR - Enter the initials of the ISS Operator.

PRO - Enter the initials of the computer operator.

HORIZONTAL SMOOTH DISC # 1

CPC # 335309

FILE	DATE	# PNTS	STATIONS	RUN #	RUN LENGTH	OPR	PRO
JUL 31 a 5	85-07-31	15	699030 to 699037	10 S	97 km	CT	CC
JUL 31 a 8	85-07-31	9	85T2080 to BCH-4	9 N	57	CT	CC
JUL 31 a 10	85-07-31	9	BCH-4 to 85T2080	9 S	57	CT	CC
JUL 31 a 11	85-07-31	15	699037 to 699030	10 N	97	CT	CC
1	2	3	4	5	6	7	

HORIZONTAL SMOOTH DISC

FILE - Enter: 1) the month - 3 letters only
2) the day
3) the letter representing the alignment identification
4) the disc file number

DATE - Enter the year, month and day

PNTS - Enter the number of points (marks and horizontal updates) for the horizontal traverse.

STATIONS - Enter the initial and final horizontal update station of the traverse.

RUN # - Enter the run (traverse) number and the direction of travel: (N - north ; S - south ; E - east , W - west).

RUN LENGTH - Enter, in kilometres, the distance of the traverse.

OPR - Enter the initials of the ISS Operator.

PRO - Enter the initials of the computer operator.

D12-1

ISS FILM LOG

CPC PROJECT: 335301

PARTY CHIEF: DOUGLAS SCOTT

DATE	ROLL	PHOTO	NEG	STATION	TRAV	OPR	PRO	LEGIBLE ON PHOTO	Sta. Confirmed on Computer O/P	DESC. OF MARKER	Agrees with Description
85-06-05	1	2	0A	60408	12	JC	DS	GEODETIC SURVEY OF CANADA TRIANGULATION STATION 60408 FOR INFORMATION WRITE DIRECTOR, OTTAWA	✓	TABLET SET FLUSH WITH CONCRETE	✓
85-06-05	1	3	1A	83R766	12	JC	DS	MANITOBA 8 R766	✓	TABLET SET IN PIPE IN CONCRETE - ABOVE GROUND LEVEL (TOO MUCH SUN)	✓
85-06-05	1	4	2A	60408	12	JC	DS	GEODETIC SURVEY OF CANADA TRIANGULATION STATION 60408 FOR INFORMATION WRITE DIRECTOR, OTTAWA	✓	TABLET SET FLUSH WITH CONCRETE	✓
85-06-05	1	5	3A	784165	16	CT	DS	TOO FAR AWAY TO READ	✓	TABLET SET IN GRASSY HOLE	✓
85-06-05	1	6	4A	83R768	16	CT	DS	TOO FAR AWAY TO READ	✓	TABLET SET IN HOLE BUT NOT FLUSH WITH GROUND.	✓
85-06-06	1	7	5A	784272	18	JC	DS	PHOTO BLURRY	✓	CANNOT SEE TARGET	✓
85-06-06	1	8	6A					NOT PRINTED - BLURRY	✓		✓
85-06-06	1	9	7A	784166	20	CT	DS	CANNOT BE READ	✓	TABLET SET IN DEEP HOLE	✓
85-06-06	1	10	8A	83R770	20	CT	DS	SURVEYS & MAPPING BRANCH 83R770	✓	TABLET SET IN HOLE BUT NOT FLUSH WITH GROUND	✓
85-06-07	1	11	9A	83R768	16	JS	DS	TARGET ON GROUND PROTRACTOR	✓	TARGET ON GROUND	✓
85-06-07	1	12	10A	83R768	16	JS	DS	SURVEYS & MAPPING BRANCH MANITOBA 83R768	✓	TABLET IN HOLE FLUSH WITH GROUND	✓

ISS FILM LOG

This form is used in conjunction with 35 mm photos of inertial update station markers to confirm that thestation number used in the data processing is the station the inertial crew used in the field.

CPC PROJECT:	The Cost and Production Control number assigned to the project.
DATE:	The date on which the photographs were taken - from ISS Daily Run Sheet.
ROLL: PHOTO: NEG:	For the 35 mm film, the Roll Number and Photo Number assigned by the field officer. Neg. is the number imprinted on the film roll negative by the manufacturer.
STATION:	The unique station number as listed in the NGDB.
TRAV.:	The traverse number assigned by the party chief - from ISS Daily Run Sheet.
OPR:	Initials of the inertial system operaator - from ISS Daily Run Sheet.
PRO:	Initials of the field office data processor.
LEGIBLE ON PHOTO:	All information that can be discerned on the photo of the marker - most importantly the marker station name or numbers.
Sta. Confirmed: On Computer O/P:	Confirmation using the final computer output that the station used in the adjustment and that positioned are the same.
DESC. OF MARKER, etc.:	When station number is missing or illegible, this column should be used to record any useful information that could be used to help confirm tthe station (e.g. iron post marker, tablet set flush with ground, etc.).
Agrees with Desc.:	Used to note whether the"DESC. OF MARKER" etc. column agrees with the NGDB description of the station.

National Geodetic Data Base

FORM 58- COMPOSITE STATION DATA CODING SHEET (GSC FIELD OFFICERS)

AGENCY ____________________

CPC PROJECT NO ______________

GENERAL — CARD CODE 5,5

STA NO (LEFT JUSTIFIED) ____ STA NAME (LEFT JUSTIFIED) ____ STA MK CLASS ____ STA MK REC (LEFT JUSTIFIED) ____ ESTAB'G AGY ____

PROV CODE ____ PHOTO ROLL NO (RIGHT JUSTIFIED) ____ PHOTO NO (RIGHT JUSTIFIED) ____ LAWNIKANIS PHOTO PIN POINTER MK 2 Y-AXIS ____ X AXIS ____

INSP'N — 5,6

INSP'N AGY ____ STA NO (LEFT JUSTIFIED) ____ YEAR OF INSPECTION ____ MKR CONDITION ____ ACCESSIBILITY T,D ____

STATION DESCRIPTION — CARD CODE 5,7

STA NO (LEFT JUSTIFIED) ____ | LINE NUMBER 0,0,0 | TEXT: M,K,R, T,Y,P,E, ', M,K,R, S,E,T,T,I,N,G, $ ◄—— MAIN MARKER

DESCRIBE BRIEFLY IN THE TEXT THE SURFACE MARKER (MKR TYPE AND MKR SETTING) AND THE SUBSURFACE MARKER AND REF. MARKERS IF APPLICABLE.

NEW DESCRIPTION .... ☐

REPLACEMENT DESCRIPTION .... ☐

LINE NUMBERS

A-BENCH MARKS (BM)

001 TO 009 BM HEADING (EN)
010 TO 249 BM TEXT (EN)

250 TO 259 BM HEADING (FR)
260 TO 499 BM TEXT (FR)

B-SURVEY MARKERS (SM)

500 TO 749 SM TEXT (EN)
951 TO 974 SM TABLE (EN)

750 TO 949 SM TEXT (FR)
976 TO 999 SM TABLE (FR)

LINE NUMBER	REFERENCE MARKERS (SPECIFY ORIENTATION - AZ/DIR/BRNG)	MKR T	MKR S	AZ/DIR/BRNG DEG	AZ/DIR/BRNG MIN	AZ/DIR/BRNG SEC	DIST (M) C X,X,X,·,X,X	DH (CM) ±,X,X,X,X

NGDB-58 (REV. MARCH- 1986) COMPILED BY ____________ DATE ________ CHECKED BY ____________ DATE ________ CARDS 33 & 34 ON REVERSE

D13-1

VERTICAL — CARD CODE 33

STA NO (LEFT JUSTIFIED) 8 | METH 31 | ORDER (N or I) 32 | DATUM FACTOR 33 | VERT SOURCE AGY 35

WTR LVL STA 40 | ELEV (M) 41 | DATE OF WTR LVL YY 51 MM 56 DD 57 | YEAR BM TIED IN 62

HORIZONTAL — CARD CODE 34

STA NO (LEFT JUSTIFIED) 8 | METH 31 | ORDER (N or S) 32 | HOR SOURCE AGY 35

LATITUDE DEG 41 MIN 44 SECONDS 47 | LONGITUDE DEG 56 MIN 63 SECONDS 63 | TOWER 72 | YR LAST TIED IN 73

CARD 55

STATION NUMBER

SEE THE NGDB USERS GUIDELINES

STATION NAME

THE NON-UNIQUE NUMBER WHICH IS STAMPED ON THE SURVEY MKR, OR THE NON-UNIQUE NUMBER OR GIVEN NAME WHICH IS ASSIGNED TO THE STATION BY THE ESTABLISHING AGENCY. IF THE UNIQUE NUMBER IS THE ONLY IDENTIFICATION WHICH IS KNOWN FOR THE STATION, CODE THIS NUMBER AS STATION NAME.

STATION MARK CLASS

1 -PERMANENT AGY MKR
2 -TEMPORARY AGY MKR
3 -STATION EVIDENCE
4 -NOT RECORDED
5 -NOT MARKED
6 -DESTROYED
7 -PERMANENT FEATURE
Y -CONTACT AGY

STATION MARK RECORDS

CODE AS MANY CHARACTERS AS APPLICABLE FROM FOLLOWING LIST.

A -PIN POINT ON AERIAL PHOTO
B -OBLIQUE PHOTO OF TARGETTED STA
C -VERTICAL PHOTO OF TARGETTED STA
D -SKETCH
E -HORIZONTAL SNAPSHOT
F -PUGGED IDENT.
X -NO STA RECORDS
Y -CONSULT AGENCY

PROVINCIAL CODE

A -NFLD	E -QUE	I -ALTA	M -US
B -NS	F -ONT	J -BC	N -FR
C -NB	G -MAN	K -NWT	P -DK
D -PEI	H -SASK	L -YT	

CARD 56

MKR CONDITION

1 -GOOD
2 -DAMAGED
3 -REPAIRED
4 -DESTROYED
5 NOT FOUND
6 -MKR DISPLACED
7 -COND UNKNOWN

MKR ACCESSIBILITY

TRANSPORTATION (T)

A PASSG'R CAR OR LT TRUCK
B -4 WHEEL DR
C BOAT
D -SEAPLANE
E -HELICOPTER
F RAILWAYS
G -OTHERS

DISTANCE WALKED (D)

1 -0 TO 50 M
2 -50 M TO 500 M
3 -0.5 KM TO 2.0 KM
4 -2.0 KM TO 5.0 KM
5 -5.0 KM AND OVER

CARD 57

MKR TYPE (T)

A ALUM TABLET
B SURVEY BOLT
C CU ROD AND OR CAP
D OR TABLET
E SPREADFOOT
F IR PIPE (HELIX BASE)
G DRILL HOLE
H IR PIPE (GSC TYPE 3)
I IR PIPE OTHER THAN F, H, P
J DLS STD POST
K IR BAR
L -NAIL
M -CHISELLED MK
N -PILE DRIVEN
O -SLEEVE TYPE
P NRC DEEP BM
Q SPIKE
R -SPLIT CAP
S -PVC MON
T -WOODEN POST
U -CAIRN
V -OTHERS (EXPLAINED IN DESCRIPTION)

SETTING CODE (S)

00 -SETTTING NOT NOTED
01 -EMBEDDED IN GROUND
02 -SET IN CONCRETE STRUCTURE
03 -SET IN TOP OF CYL CONC MON
04 SET IN TOP OF SQUARE MON
05 ATTACHED TO A METAL ROD
06 SET IN BEDROCK
07 -SET IN BOULDER
08 -X CROSSED CHISEL LINES

DISTANCE CODE (C)

H HORIZONTAL DISTANCE
S SLOPE DISTANCE
A -SCALED DIST, PACED DIST, ETC

DIFF IN HEIGHT (DH)

RIGHT JUSTIFIED

CARD 33 *

METHOD

1 SPIRIT LEVELS
2 SIMULTANEOUS TRIG LEVELS
3 -NON SIMULTANEOUS TRIG LEVELS
4 -AIRBORNE TRIG LEVELS
5 -OTHER METHODS USED IN MAPPING CONT SURVEYS. INCLUDE GEM, STADIA, APR, BAROMETER LEVELLING, ETC.
6 INERTIAL LEVELLING
7 -DOPPLER DERIVED
8 GPS DERIVED
I -INTERPOLATED FROM CONTOURS
X -NO ELEV ESTABLISHED
Y -CONSULT AGY

ORDER

I -INTERPOLATED VALUES
N -NON ADJUSTED FIELD VALUES

DATUM FACTOR

P -DATUM IS PRIMARY VERT. CONTROL NET.
A DATUM VALID TO ± 0.3 METRE
B DATUM VALID TO ± 1.0 METRE
C -DATUM VALID TO ± 3.0 METRES
D -DATUM VALID TO ± 10 METRES
E -UNIQUE COND. EXPLAINED IN DESCRIPTION
X -NOT EVALUATED

WTR LVL STATION

W -STA. IS AT WATER LVL
BLANK -STA. IS NOT AT WATER LVL

CARD 34

METHOD

A -DOPPLER POSITIONING
B -TRIANGULATION TRILATERATION
C -BASELINE TRIANGULATION
D -TRILATERATION
E -ELECTRONIC TRAVERSING
F -CHAIN TRAVERSE
G -AIR TRILATERATION (Shoran, Aerodist)
H -ASTRONOMIC POSITION
P -INERTIAL SURVEY SYSTEM
Q -PHOTOGRAMMETRIC POSITION
R -GLOBAL POSITIONING SYSTEM
S -SCALED POSITION, STADIA, AERODIST PHOTO FIXING, ETC.

ORDER

N NON ADJUSTED FIELD VALUES
S SCALED VALUES
INDICATE THE SCALED VALUES TO NEAREST 6 SEC.

TOWER

A TOWER USED
BLANK NO TOWER USED

*Notes

-THE AGY CODE CORRESPONDING TO GSC IS 100

-FOR ADDITIONAL INFO RE: STATION NUMBERING, STATION NAMING, STA DESCRIPTION EDITING AND DATA CODING IN GENERAL CONSULT THE "NGDB USERS GUIDELINES-DATA CODING INSTRUCTIONS AND SPECIFICATIONS"

D13-2

STATION LIST

GEODETIC SURVEY OF CANADA

Pg___ of___

PARTY CHIEF ______________ PROJECT NAME ______________ SURVEY AGENCY ______________

CPC NUMBER ______________ AREA OR PROV ______________ YEAR OF SURVEY ______________

STATION NUMBER	STATION NAME	MAP SHEET	card 34 APPROX LAT	APPROX LONG	*HORIZ METHOD	card 33 APPROX ELEVATION	*VERT METHOD	card 56 *INSPECTION MKR COND	ACCESS T	D	card 55 STATION RECORDS	card 57 MKR TYPE T	S	REMARKS

D14-1

NGDB 86-1 *NOTE: CODES ARE ON THE BACK

card 34
HORIZ METHOD

A - SATELLITE POSITIONING (Doppler)
B - TRIANGULATION - TRILATERATION
C - BASELINE TRIANGULATION
D - TRILATERATION
E - ELECTRONIC TRAVERSING (EDM)
F - CHAIN TRAVERSE
G - AIR TRILATERATION (Shoran, Aerodist)
H - ASTRONOMIC POSITION
P - INERTIAL SURVEY SYSTEM (ISS)
Q - PHOTOGRAMMETRIC POSITION
R - GLOBAL POSITIONING SYSTEM (GPS)
S - SCALED POSITION (Stadia, Aerodist, Photo fixing, etc.)

card 55
STATION RECORDS

A - PIN PHOTO ON AERIAL PHOTO
B - OBLIQUE PHOTO OF TARGETTED STATION
C - VERTICAL PHOTO OF TARGETTED STATION
D - SKETCH
E - HORIZONTAL SNAPSHOT
F - PUGGED IDENT
X - NO STATION RECORDS
Y - CONSULT AGENCY

card 57
MARKER TYPE
(T)

A - ALUM TABLET
B - SURVEY BOLT
C - CU ROD AND BR CAP
D - BR TABLET
E - SPREADFOOT
F - IR PIPE (HELIX BASE)
G - DRILL HOLL
H - GSC TYPE 3 MON
I - IR PIPE OTHER THAN F, H, P,
J - DLS STD POST
K - IR BAR
L - NAIL
M - CHISELLED MARK
N - PILE DRIVEN
O - SLEEVE TYPE
P - NRC DEEP BM
Q - SPIKE
R - SPLIT CAP
S - PVC MON
T - WOODEN POST
U - CAIRN
V - OTHERS (EXPLAINED IN DESCRIPTION)

card 57
MARKER SETTING
(D)

00 - SETTING NOT NOTED
01 - EMBEDDED IN GROUND
02 - SET IN CONCRETE STRUCTURE
03 - SET IN TOP OF CYL CONC MON
04 - SET IN TOP OF SQUARE MON
05 - ATTACHED TO A METAL ROD
06 - SET IN BEDROCK
07 - SET IN BOULDER
08 - X CROSSED CHISEL LINES

card 33
VERT METHOD

1 - SPIRIT LEVELS
2 - SIMULTANEOUS TRIG LEVELS
3 - NON-SIMULTANEOUS TRIG LEVELS
4 - AIRBORNE TRIG LEVELS
5 - OTHER METHODS USED IN MAPPING CONTROL SURVEYS (Gem, Stadia, APR, Barometer levelling, etc.)
6 - INERTIAL LEVELLING (ISS)
7 - DOPPLER DERIVED
8 - GLOBAL POSTIONING SYSTEM (GPS)
I - INTERPOLATED FROM CONTOURS
X - NO ELEVATION ESTABLISHED
Y - CONSULT AGENCY

card 56
MARKER CONDITION

1 - GOOD
2 - DAMAGED
3 - REPAIRED
4 - DESTROYED
5 - NOT FOUND
6 - MARKER DISPLACED
7 - CONDITION UNKNOWN

card 56
STATION ACCESSIBILTY
(D) WALKING DISTANCE

1 - 0 TO 50 M
2 - 50 M TO 500 M
3 - 0.5 KM TO 2.0 KM
4 - 2.0 KM TO 5.0 KM
5 - 5.0 KM AND OVER

card 56
STATION ACCESSIBILTY
(T) TRANSPORTATION CODE

A - PASSENGER CAR OR LIGHT TRUCK
B - 4-WHEEL DRIVE
C - BOAT
D - SEAPLANE
E - HELICOPTER
F - RAILWAYS
G - OTHERS

National Geodetic Data Base

FORM 59 - STATION DESCRIPTION SKETCH

AGENCY______________________

CPC PROJECT NO______________

SKETCHES, COMMENTS, ETC.

N

STA NUMBER

STA NAME

PROVINCE ______________

DRAW ALL SKETCHES ROUGHLY TO SCALE IN BLACK INK. LIGHT PENCILLED SKETCHES WILL NOT REPRODUCE WHEN MICROFILMED.

INDICATE ALL DISTANCES IN METRIC UNITS.

THE STATION NUMBER IS THE NGDB ASSIGNED NUMBER. THE STATION NAME IS THE NON-UNIQUE NUMBER WHICH IS STAMPED ON THE SURVEY MARKER, OR THE NON-UNIQUE NUMBER OR GIVEN NAME WHICH IS ASSIGNED TO THE STATION BY THE ESTABLISHING AGENCY.

DRAWN BY______________________ DATE______________

DOPPLER AUTOMATIC STATION

Page: ____ of ____

ANTENNA ECCENTRIC: NO ☐ YES ☐ SKETCH ON BACK

OBSTRUCTION OVER 15°: NO ☐ YES ☐ ① ____

SOURCE OF INTERFERENCE: ② ____

ALTERNATIVE MET SOURCE: ③ ____

ELEVATION TIE: NO ☐ YES ☐ TYPE: ④ ____

TYPE OF GROUND: ____

OPERATOR(S): ____

STA. NAME ____

STA. NUMBER ⑤ ____

MAP SHEET ____

LAT. ⑥ ____ ° ' "

LONG. ____ ° ' "

ELEV. ____ M

YEAR ____

ANT. # ____ RCVR. # ____ DCU. # ____ HT. ANT. BASE ⑦ ____ M ESU # ____

ESU COEFF. ⑧ A: ____ . ____ B: ____ . ____ T_0: ____ . ____

EQUIPMENT CHANGE ⑨: NO ☐ YES ☐ SEE PAGE ____

	1	2 ㉖	3	4
DATE/TIME (LOCAL)				
T°C: WET/DRY ⑩				
ALT #/RDG (FT) ⑪				
CORR/CORR'D RDG (FT) ⑫				
MET: T°C/P(mb)/RH ⑬				
ESU: T°C/P(mb)/RH ⑭				
BATTERY: EXT/INT ⑮				
COUNTERS ZEROED ⑯	YES ☐ NO ☐	YES ☐ NO ☐	YES ☐ NO ☐	
CASSETTE # ⑰				
FIRST ALERT: SAT/DAY/TIME ⑱				
LAST ALERT: SAT/DAY/TIME ⑲				
STATUS: FIXES/STORED ⑳				
OSC. STABILITY ㉑				
MJV D.C. #/DCU # ㉒				
DCU COUNTER: RAW/MJV ㉓				

LAST FIX: LAT. ____ ° ' . " ㉔ LONG ____ ° ' . " HT ____ . ____ m

CARTESIAN COORDS: X ㉕ ____ Y ____ Z ____

REMARK(S): ____

PHC-76 (1985)

DOPPLER AUTOMATIC STATION

This form was designed specifically for the CMA 761 receiver for use by the observer and the person post-processing the data.

Numbers below correspond to those on sample form.

1. Refers to obstructions between the satellites and antenna.
2. Refers to radio transmitters, any frequency, that may interfere.
3. If ESU is absent or fails, list the closest source of meteorological information, i.e. another Doppler station, airport weather station, etc.
4. Marker elevation by spirit levels, altimeter, trigonometric heighting or ISS.
5. This is the NGDB number.
6. Lat., Long., Elev. - use the best available, these are entered at the set up of station, nearest minute of lat. and long. are sufficient if corresponding standard deviations are entered in the receiver. The object is to give the receiver software coordinates so that the first pass can be computed. Once one "fix" is achieved, the computed coordinates are used for succeeding passes.
7. This is the height, to the nearest centimetre, from the marker to the antenna base.
8. These coefficients are unique to each ESU. They effect the atmospheric pressure readout only, not the temperature or relative humidity.
9. If the equipment is changed start another page for clarity. This is particularily important if a receiver is changed.
10. To be taken with a conventional sling or electric psychrometer.
11. Refers to Wallace and Tiernan survey altimeter.
12. Correction is from a calibration graph, then subtract 1000 feet to get pressure altitude in feet.
13. Record the psychrometers dry bulb temperature, the station pressure converted to millibars and the relative humidity as determined from tables.
14. Corresponding readings from ESU - the object is to spot any malfunction of ESU.
15. List voltages of receiver (external and internal) batteries, nominally 12 volts.
16. Refers to receiver "fix" counters and "slave" counters, applies when changing cassettes or stations.
17. A unique number is used for each cassette.
18. This is the next pass as computed by the receiver software. This is also the last line recorded at the beginning of an observing period.
19. This shows the last pass that the receiver software planned to track.
20. Shows the number of passes successfully majority-voted and stored in the slave and the number of these passes on which a 3D position fix was achieved.
21. Indicates oscillator stability as computed from the last fix.
22. Indicates cassette number of the slave data, corresponds to item 17 for the raw data. Second space is for the serial number of the DCU used to dump the slave data.
23. As an aid to those people later reading the data the counter reading on the DCU at the end of the raw data and the slave data is recorded.
24. The latitude, longitude and ellipsoid height as computed by the receiver is recorded.
25. Cartesian coordinates can be used as starting coordinates for post processing.
26. Additional columns are used for successive cassettes, if required. Note that items 19-25 and items 10 to 15 at the top of column 2 are completed when removing a cassette. Then items 16-18, column 2, are completed if starting a second cassette.

APPENDIX E

STEPS TO CONVERT LOCAL MEAN TIME TO LOCAL SIDEREAL TIME AND TO COMPUTE THE RATE AND THE LEVEL CORRECTION

STAR ☑ OR SUN ☐

STATION NEPEAN-2 ECC REFERENCE OBJECT (RO) COURT SET NO 1

LATITUDE 45°25′46″ COMPUTED BY J. LAFRANCE

LONGITUDE 75°42′05″ CHECKED BY L.J. HENNESSEY

RIGHT ASCENSION $2^h 14^m 38.^s3$ DATE OBSERVED 1985 MAY 13/14

DECLINATION 89°11′45.″4

LEVEL VALUE 1.″1 / division = d

ALTITUDE OF CO 44°40′22″ = h from SALS

RADIO ANNOUNCEMENT AT THE BEGINNING OF OBSERVATION PROGRAM 4 h 39 m 00 s (UTC)

RADIO ANNOUNCEMENT AT THE END OF OBSERVATION PROGRAM 7 h 30 m 00 s (UTC)

CLOCK SHOWS AT THE BEGINNING OF OBSERVATION PROGRAM 15 h 03 m 40 s 5 (UT) OR (LST1) −0.ˢ4

CLOCK SHOWS AT THE END OF OBSERVATION PROGRAM 17 h 55 m 08 s 6 (UT) OR (LST2) −0.ˢ4

	AT BEGINNING		AT END
UTC (IF EST ADD 5 HRS) $23^h 39^m 00^s + 5^h$	4 h 39 m 00 s		7 h 30 m 00 s
CORRECTION FOR (UTC − UT) FROM CODED SIGNAL	−0 s 4		−0 s 4
FROM SUN TABLE (R AT 0^hUT)	15 h 26 m 42 s 9		15 h 27 m 42 s 1
FROM SUN TABLE (CORRECTION TO R FOR HRS, MIN)	h m 45 s 9		h m 14 s 8
GREENWICH SIDEREAL TIME	GST1 20 h 06 m 28 s 4	GST2	22 h 57 m 56 s 5
LONGITUDE IN TIME	5 h 02 m 48 s 3		5 h 02 m 48 s 3
LOCAL SIDEREAL TIME	15 h 03 m 40 s 1		17 h 55 m 08 s 2
CLOCK TIME (LST)	15 h 03 m 40 s 1		17 h 55 m 08 s 2
ΔT	m 00 s 0		m 00 s 0

HCR (L) ON CO 151° 30′ 17.″2 HCR (L) ON RO 0° 00′ 00.″0

HCR (R) ON CO 331 30 54.7 HCR (R) ON RO 180 00 07.1

MEAN HCR CO 151° 30′ 36.″0 MEAN HCR RO 0° 00′ 03.″6

CLOCK TIME (L) ON CO $15^h 32^m 14.^s1$ CLOCK TIME (R) ON CO $15^h 36^m 57.^s1$

MEAN CLOCK TIME (L & R) $15^h 34^m 35.^s6$ (UTC − UT) −0.ˢ4

COOR'D CLOCK $15^h 34^m 35.^s2$ ΔT NIL

MEAN HCR RO − MEAN HRC CO = MEAN ANGLE

= 208° 29′ 27.″6

$$\text{RATE} = \frac{\Delta GST - \Delta LSC}{\Delta LSC} = \frac{(GST2 - GST1) - (LST2 - LST1)}{(LST2 - LST1)} = \frac{(2^h 51^m 28.^s1) - (2^h 51^m 28.^s1)}{(2^h 51^m 28.^s1)} = 0 \text{ s/m}$$

LEVEL

	EAST		WEST
E′	71.8	W′	28.2
E	30.2	W	73.8
(E−E′)	−41.6	(W−W′)	+45.6 = +4.0 LEVEL DIFFERENCE (LD)

LEVEL CONVENTION FOR SIGN

The primed letters refer to the readings taken in the position in which the numbering increases towards the east.

LEVEL CORRECTION (LC)

$$LC = \frac{d}{4} [LD] \tan h = +1.''1$$

AZIMUTH BY HOUR ANGLE METHOD (MEAN TIME OR SIDEREAL TIME)

SKETCH

PROJECT PARLIAMENT HILL PROJECT		LOCATION OTTAWA (PARLIAMENT HILL)	
PARTY CHIEF O.J. LAFRANCE	LATITUDE (φ) 45° 25' 46"	LONGITUDE (λ) 75° 42' 05" OR 5h 02m 48s.3	STATION NEPEAN-2 ECC.
REFERENCE OBJECT (RO) COURT	INSTRUMENT		TIME ZONE 5h
CELESTIAL OBJECT (CO) OBSERVED POLARIS	TIME SIGNAL TIME BEGINNING 23h 39m 00s.0 END 02h 30m 00s.0		WATCH TIME (SIDEREAL OR MT) BEGINNING 15h 03m 40s.5 END 17h 55m 08s.6
DATE 1985 MAY 13/14	OBSERVER O.J. LAFRANCE	TEMPERATURE WEATHER GOOD	PRESSURE

	SET 1						SET 2						SET 3					
	TIME			HORIZONTAL ANGLE			TIME			HORIZONTAL ANGLE			TIME			HORIZONTAL ANGLE		
MEAN	HRS	MIN	SEC	°	'	"	HRS	MIN	SEC	°	'	"	HRS	MIN	SEC	°	'	"
	00	09	50.0	208	29	27.6												

	STAR OBSERVATION			SET 1			SET 2			SET 3		
				HRS	MIN	SEC	HRS	MIN	SEC	HRS	MIN	SEC
1	MEAN TIME OF OBSERVATION			00	09	50.0						
2	WATCH CORRECTIONS (FAST −, SLOW+)	±				0.0						
3	TIME ZONE CORRECTION	±		5								
4	CORRECTION FOR (UTC − UT) FROM CODED TIME SIGNAL OR BULLETIN		−			0.4						
5	UT OF OBSERVATION (1 + 2 + 3 + 4)			5	09	49.6						
6	R AT 0h UT R FROM SALS TABLES			15	26	42.9						
7	CORRECTION FOR R, FOR HRS. MIN.	±	+			51.0						
8	(6 + 7) (GST − UT) AT EPOCH OF OBSERVATION			15	27	33.9						
9	(5 + 8) GST			20	37	23.5						
10	LONGITUDE (WEST −)			5	02	48.3						
11	(9 − 10) LST			15	34	35.2						
12	RA OF STAR OR SUN		−	2	14	38.3						
13	LHA (11 + 12) = t (IN TIME)			13	19	56.9						
14	LHA IN ARC = t (IN ARC)			199	59	13.5						

		SET 1			SET 2			SET 3		
		°	'	"	°	'	"	°	'	"
t	±	199	59	13.5						
φ	±	45	25	46						
δ	±	89	11	45.4						
SIN t	±	−0.34180829								
COS t	±	−0.93976970								
SIN φ	±	0.71238679								
COS φ	±	0.70178705								
TAN δ	±	71.2538056								
TAN A	±	0.00674518								
A EAST or WEST		00	23	11.3						
AZIMUTH OF CO		00	23	11.3						
HCS (RO)		00	00	03.6						
HCR (CO)		151	30	36.0						
AZIMUTH TO RO		208	52	38.9						

−0.34180829

$$\text{TAN A} = -\frac{\text{SIN } t}{\text{COS } \phi \text{ TAN } \delta - \text{SIN } \phi \text{ COS } t}$$

50.00499791 + 0.66947952

IF CELESTIAL OBJECT (CO) EAST

THEN AZIMUTH = A OR 180° − A

IF CELESTIAL OBJECT (CO) WEST

THEN AZIMUTH = 360° − A OR 180° + A

MEAN AZIMUTH	°	'	"

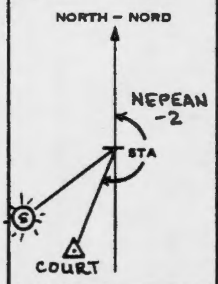
COMPUTED BY	DATE	CHECKED BY	DATE
C.G.V.	85-11-27	W.R.B.	85-11-27

GEODETIC SURVEY
LEVÉS GÉODÉSIQUES

AZIMUTH BY HOUR ANGLE METHOD (MEAN TIME)
AZIMUT PAR MÉTHODE D'ANGLE HORAIRE (TEMPS MOYEN)

(USING "E")

DIAGRAM – SCHÉMA



PROJECT – PROJET: PARLIAMENT HILL PROJECT		LOCATION – LIEU: OTTAWA (PARLIAMENT HILL)	
PARTY CHIEF – CHEF D'ÉQUIPE: L.J. HENNESSEY	LATITUDE (ϕ): 45° 25' 46"	LONGITUDE (λ): 75° 42' 05"	STATION: NEPEAN-2
REFERENCE OBJECT (RO) – OBJET VISÉ (OV): COURT	INSTRUMENT: WILD T-2 #145500		TIME ZONE – ZONE HORAIRE: E.S.T.
CELESTIAL BODY OBSERVED (☉) CORPS CÉLESTE OBSERVÉ (☉): SUN	TIME SIGNAL TIME HEURE DU SIGNAL HORAIRE: CHU 15h 09m 00.0 (START) WWV 21 33 00.0 (END)		WATCH TIME – HEURE DE MONTRE UT: 20h 01m 00.2 / 21 25 02.9
DATE: 85-01-29	OBSERVER – OBSERVATEUR: L.J. HENNESSEY		WEATHER – MÉTÉO: CLEAR & COLD

	SET 1 - SÉRIE 1						SET 2 - SÉRIE 2						SET 3 - SÉRIE 3					
	TIME – HEURE			HORIZ. ANGLE HORIZ.			TIME – HEURE			HORIZ. ANGLE HORIZ.			TIME – HEURE			HORIZ. ANGLE HORIZ.		
	HRS.	MIN.	SEC.	°	'	"	HRS.	MIN.	SEC.	°	'	"	HRS.	MIN.	SEC.	°	'	"
MEAN MOYENNE ▶	15	13	14.4	14	26	16.5												

	SOLAR OBSERVATION SOLAIRE		SET 1 - SÉRIE 1 HRS.	MIN.	SEC.	SET 2 - SÉRIE 2 HRS.	MIN.	SEC.	SET 3 - SÉRIE 3 HRS.	MIN.	SEC.
1	MEAN TIME OF OBSERVATION / HEURE MOYENNE D'OBSERVATION	EST	15	13	14.4						
2	WATCH CORRECTION (FAST -, SLOW +) / CORRECTION DE MONTRE (EN AVANCE - EN ARRIÈRE +)	⊕		07	59.2						
3	TIME ZONE CORRECTION / CORRECTION POUR ZONE HORAIRE	±	5								
4	UT OF OBSERVATION / TU D'OBSERVATION (1 + 2 + 3)		20	21	13.6						
5	EQT AT 0h UT / EQT À 0h TU	"E" + 18h	11	46	45.9						
6	UT X VAR. EQT PER HOUR / TU X VAR. DE ÉQT PAR HEURE	±		-	1.0						
7	(5 + 6) EQT AT TIME OF OBS. / (5 + 6) ÉQT À L'HEURE D'OBS.		11	46	44.9						
8	(4 + 7) GAT / (4 + 7) TSVG		32	07	58.5						
9	GHA IN TIME (GAT - 24 h) / AHG EN TEMPS (TSVG - 24 h)		8	07	58.5						
10	GHA IN ARC / AHG EN ARC		121°	59'	38"						
11	LONGITUDE (WEST -) / LONGITUDE (OUEST -)		75	42	05						
12	LHA (10 + 11) = t / AHL (10 + 11) = t		46	17	33						

		SET 1 - SÉRIE 1 °	'	"	SET 2 - SÉRIE 2 °	'	"	SET 3 - SÉRIE 3 °	'	"
t	±	46	17	33						
ϕ	±	45	25	46						
δ	±	-17	46	24						
SIN t	±	0.7228767								
COS t	±	0.6909770								
SIN ϕ	±	0.7123868								
COS ϕ	±	0.7017870								
TAN δ	±	-0.3205516								
- TAN A	±	+1.0079126								
A (E OR W) / A (E OU O)		45	13	33						
AZIMUTH OF ☉ / AZIMUT DE ☉		225	13	33						
HCR (RO) / LCH (OV)		00	00	18.5						
HCR ☉ / LCH ☉		14	26	16.5						
AZIMUTH TO RO / AZIMUT À OV		210	47	35						

$$-\tan A = \frac{\sin t}{\cos\phi \tan\delta - \sin\phi \cos t}$$

IF LHA IS GREATER THAN 180°, SUBTRACT FROM 360° AND REVERSE SIGN. CHECK SIGNS AND QUADRANTS BY USE OF THE DIAGRAM.

$$-\tan A = \frac{\sin t}{\cos\phi \tan\delta - \sin\phi \cos t}$$

SI L'AHL EST PLUS GRAND QUE 180°, LE SOUSTRAIRE DE 360° ET RENVERSER LE SIGNE. VÉRIFIER LES SIGNES ET QUADRANTS À L'AIDE DU SCHÉMA.

MEAN AZIMUTH / AZIMUT MOYEN ▶ ° ' "

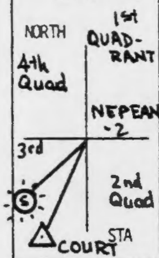
COMPUTED BY – CALCULÉ PAR	DATE	CHECKED BY – VÉRIFIÉ PAR	DATE
L.J. HENNESSEY	85/10/28	O.J. LAFRANCE	85/11/25

Form 77A20

E4

AZIMUTH BY HOUR ANGLE METHOD (MEAN TIME OR SIDEREAL TIME) (USING "R")

SKETCH



PROJECT PARLIAMENT HILL		LOCATION OTTAWA (PARLIAMENT HILL)	
PARTY CHIEF L.J. HENNESSEY	LATITUDE (φ) 45° 26' 46"	LONGITUDE (λ) 75° 42' 05" OR 5h 02m 48s.3	STATION NEPEAN-2
REFERENCE OBJECT (RO) COURT	INSTRUMENT WILD T2 # 145500		TIME ZONE 5h
CELESTIAL OBJECT (CO) OBSERVED SUN	TIME SIGNAL TIME CHU BEGINNING 15h 09m 00s; WWV END 21h 33m 00s	WATCH TIME (SIDEREAL OR MT) BEGINNING 20h 01m 00s.2; END 21h 25m 02s.9	
DATE 85-JAN-29	OBSERVER L.J. HENNESSEY	TEMPERATURE −8.2°C clear & cold	PRESSURE 1011.8 mb

	SET 1						SET 2						SET 3					
	TIME			HORIZONTAL ANGLE			TIME			HORIZONTAL ANGLE			TIME			HORIZONTAL ANGLE		
MEAN	HRS	MIN	SEC	°	'	"	HRS	MIN	SEC	°	'	"	HRS	MIN	SEC	°	'	"
	15	13	14.4	14	26	16.5												

	STAR OBSERVATION		SET 1 HRS	SET 1 MIN	SET 1 SEC	SET 2 HRS	SET 2 MIN	SET 2 SEC	SET 3 HRS	SET 3 MIN	SET 3 SEC
1	MEAN TIME OF OBSERVATION		15	13	14.4						
2	WATCH CORRECTIONS (FAST −, SLOW+)	+		7	59.4						
3	TIME ZONE CORRECTION	+	5								
4	CORRECTION FOR (UTC − UT) FROM CODED TIME SIGNAL OR BULLETIN			−	00.2						
5	UT OF OBSERVATION (1 + 2 + 3 + 4)		20	21	13.6						
6	R AT 0h UT R FROM SALS TABLES		8	35	42.1						
7	CORRECTION FOR R, FOR HRS. MIN.	+			23.2						
8	(6 + 7) (GST − UT) AT EPOCH OF OBSERVATION		8	36	05.3						
9	(5 + 8) GST		28	57	18.9						
10	LONGITUDE (WEST −)		5	02	48.3						
11	(9 − 10) LST		23	54	30.6						
12	RA OF STAR OR SUN (R − E) = 8h 36m 05s.3 − 11h 46m 44s.9		20	49	20.4						
13	LHA (11 + 12) = t (IN TIME)		3	05	10.2						
14	LHA IN ARC = t (IN ARC)		46°	17'	33"						

		SET 1 °	SET 1 '	SET 1 "	SET 2 °	SET 2 '	SET 2 "	SET 3 °	SET 3 '	SET 3 "
t	±	46	17	33						
φ	±	45	26	46						
δ	±	−17	46	24						
SIN t	±	0.7228767								
COS t	±	0.6909770								
SIN φ	±	0.7123868								
COS φ	±	0.7017870								
TAN δ	±	−0.3205516								
TAN A	±	1.0079126								
A EAST or WEST		45	13	33						
AZIMUTH OF CO 3rd Quad.		225	13	33						
HCS (RO)		00	00	18.5						
HCR (CO)		14	26	16.5						
AZIMUTH TO RO		210	47	35						

$$\tan A = -\frac{\sin t}{\cos\phi\tan\delta - \sin\phi\cos t}$$

0.7228767

−0.2249589 − 0.4922429

−0.7172018

IF CELESTIAL OBJECT (CO) EAST
THEN AZIMUTH = A OR 180° − A

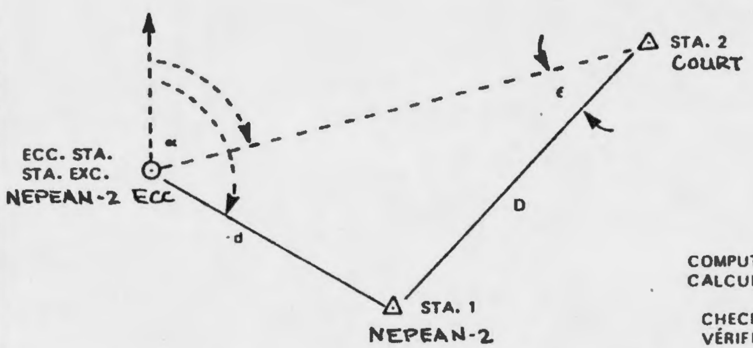
IF CELESTIAL OBJECT (CO) WEST
THEN AZIMUTH = 360° − A OR 180° + A

MEAN AZIMUTH	°	'	"

COMPUTED BY	DATE	CHECKED BY	DATE
L.J. HENNESSEY	85/10/28	JACQUES LAFRANCE	85/11/25

ECCENTRICITY AND AZIMUTH REDUCTION

RÉDUCTION POUR EXCENTRICITÉ ET AZIMUT



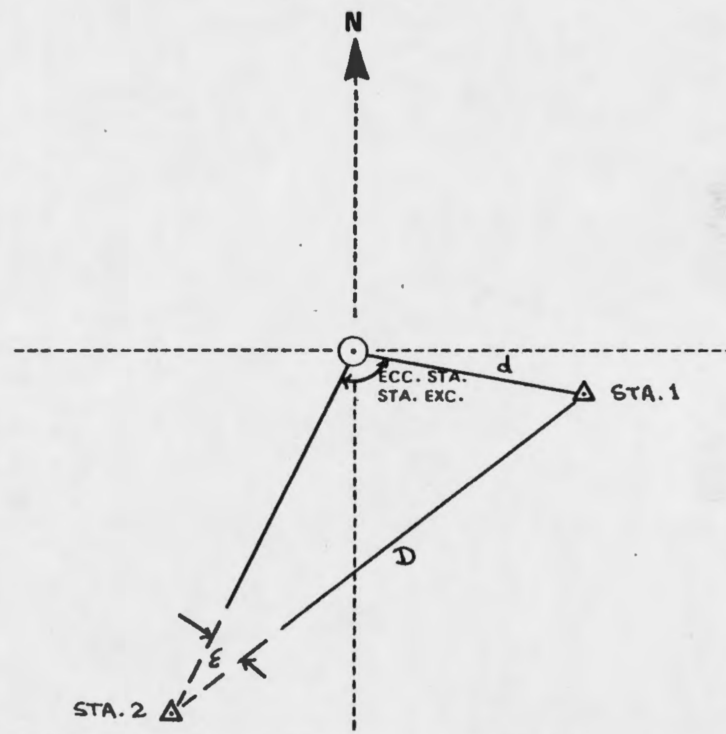
STATION 1 NEPEAN-2

STATION 2 COURT

COMPUTED BY / CALCULÉ PAR J. LAFRANCE

CHECKED BY / VÉRIFIÉ PAR C. G. VIGNEAULT

SKETCH – CROQUIS



(1)	LATITUDE (φ)	45° 25′ 46″
(2)	SIN φ	0.71239
(3)	ECC. DISTANCE / DISTANCE EXC. (d)	d = ft/pds 29.370 m
(4)	AZ. OF ECC. LINE / AZ. DE LA LIGNE EXC. (α)	102° 37′ 19.″4
(5)	IF ECC. STA. IS EAST OF STA. 1 / SI LA STA. EXC. EST À L'EST DE LA STA. 1 α = 180 − (4)	N/A
(6)	PARALLEL VALUE OF 1″ / VALEUR DU PARALLÈLE DE 1″	ft/pds 21.736 m
(7)	MERIDIAN VALUE OF 1″ / VALEUR DU MÉRIDIEN DE 1″	ft/pds 30.872 m
(8)	$\Delta\lambda = \frac{(3)[\sin\alpha]}{(6)}$	Δλ = − 1.″319
(9)	$\Delta\phi = \frac{(3)[\cos\alpha]}{(7)}$	Δφ = − 0.″208
(10)	AZ. OF ECC. STA. TO STA. 2 / AZ. DE LA STA. EXC. À LA STA. 2	208° 52′ 39.″0
(11)	(4) − (10)	106° 15′ 19.″6
(12)	DIST. STA. 1 TO STA. 2 / DIST. STA. 1 À STA. 2	846.096 m
(13)	$\sin\epsilon = \frac{(3)\sin(11)}{(12)}$	sin ε =
(14)	ε	ε = 1° 54′ 35.″0
(15)	Conv. = (8) × (2)	CONVERGENCE = + 0.″9

	Δλ	Δφ	CONV.
ECC. STA. NE OF STA. 1 / STA. EXC. N-E DE STA. 1	+	−	−
ECC. STA. SE OF STA. 1 / STA. EXC. S-E DE STA. 1	+	+	−
ECC. STA. SW OF STA. 1 / STA. EXC. S-O DE STA. 1	−	+	+
ECC. STA. NW OF STA. 1 / STA. EXC. N-O DE STA. 1	−	−	+

NOTE: APPROPRIATE SIGN MUST BE GIVEN TO AZIMUTH CORRECTION
À NOTER: APPROPRIER LE SIGNE À LA CORRECTION D'AZIMUT

76 A7

E5

STAR AZIMUTH (UNIVERSAL TIME)

1. IDENTIFICATION

Code Name: STARAZ Date: January 1982
Programme Number: GH-D-31 85-10-23 Rev'd.
Programmer: G.A. Corcoran, revised by C.G. Vigneault
Calculator: Texas Instruments TI-59 with PC-100A Printer

2. SYNOPSIS

Given the Universal Time of observation (UT) corrected for rate etc., the latitude and longitude of the observing station, and from the catalogue sidereal time at 0^h UT (R or Right Ascension of Mean Sun [RAMS] + 12^h), right ascension and declination of the star in an Almanac, this programme will calculate the azimuth of the star (hour angle method). With the HCR's on the star and on the mark known, the observations can be reduced to the azimuth of the mark.

This program was designed for lower-order azimuth determination.

3. FORMULAE

The azimuth of a star is given by:

$$\text{Tan } A = \frac{-\sin t}{\cos \phi \tan \delta - \sin\phi\cos t}$$

where

A = azimuth of the star

t = hour angle of the star = LST - α

α = right ascension of the star

δ = declination of the star

ϕ = latitude of the observation station

LST = local sidereal time

The azimuth of a star may be reduced to a mark by:

$$AZM = A + HCR^* - HCRM$$

where

AZM = azimuth of mark

HCR* = horizontal circle reading on star

HCRM = horizontal circle reading on mark

4. **GENERAL** .

Any observation which requires corrections, must be corrected before being entered in the program.

CELESTIAL OBSERVATION FOR AZIMUTH BY ALTITUDE METHOD

STAR [ ] or SUN [✓]

STATION	NEPEAN-2	RO	COURT
DATE	1985 JAN 29	LATITUDE	45° 25' 46"
OBSERVER	L.J. HENNESSEY	LONGITUDE	75° 42' 05"
TEMPERATURE	−8.°2 C	PRESSURE	1011.8 mb

CLAMP	TIME LMT OR UT	HCR (RO)	HCR (CO)	VCR (CO)
L	15h 11m 56.s9	00° 00' 15."0	14° 23' 35."0	75° 22' 11"
R	15 14 31.9	180 00 22.0	194 28 58.0	283 47 47
MEAN VALUES	15h 13m 14.s4	00° 00' 18."5	14° 26' 16."5	Z = 75° 47' 12" h = 14 12 48

LOCAL MEAN TIME (LMT)	15h 13m 14.s4	HCR (RO)	00° 00' 18."5
CLOCK CORRECTION	+ 7m 59.s4	HCR (CO)	14 26 16.5
(UTC - UT) CORRECTION	− 0.s2	HORIZONTAL ANGLE	345° 34' 02."0
ZONE CORRECTION	+ 5h		
UT	20h 21m 13.s6		
δ at 0h UT	−18° 00' 06"	h (CO) = 90° - Z =	14° 12' 48."0
VARIATION FOR HRS & MIN	+ 13' 42"	REFRACTION CORRECTION =	− 4' 01."0
		PARALLAX =	+ 08."5
δ AT OBSERVATION	− 17° 46' 24"	CORRECTED ALTITUDE = h	14° 08' 55."5
SIN δ	−0.3052521	COS A =	−0.7044666
SIN φ	0.7123868	IF EAST, A(CO) =	
COS φ	0.7017870	IF WEST,	360°00'00"
SIN h	0.2444401	- A =	134° 47' 11."3
COS h	0.9696644	A (CO) =	225° 12' 48."7

$$\text{COS A} = \frac{\text{SIN } \delta - \text{SIN } \phi \text{ SIN h}}{\text{COS } \phi \text{ COS h}}$$

IF CELESTIAL OBJECT (CO) EAST
THEN AZIMUTH = A OR 180° - A

IF CELESTIAL OBJECT (CO) WEST
THEN AZIMUTH = 360° - A OR 180° + A

A (CO)	225° 12' 48."7
HORIZONTAL ANGLE	345 34 02.0
A (RO)	210° 46' 50."7

SUN AZIMUTH - ALTITUDE METHOD

1. IDENTIFICATION

Code name: SUNAZ-H — Date: 1981-01-28
Programme Number: GH-D-27 — 85-10-23 Rev'd
Programmer: G.A. Corcoran, revised by C.G. Vigneault
Calculator: Texas Instruments TI-59 with PC-100A Printer

2. SYNOPSIS

Given the latitude of the observing station, the altitude of the celestial object (CO), corrected for refraction (and parallax if sun observed, etc.), the HCR's on the celestial object and reference mark or reference object (RO), this programme will compute the azimuth of the celestial object, as well as the azimuth of the mark. The declination is obtained from an ephemeris such as "Star Almanac for Land Surveyors" (SALS).

3. FORMULAE

When the time of an observation is not precisely known, the altitude method for azimuth is preferred. The formula for machine computation is:

$$\cos A = \frac{\sin \delta - \sin h \sin \phi}{\cos h \cos \phi}$$

where δ = declination of the CO, (+) if north, (-) if south
h = altitude of the CO
ϕ = latitude of the station
A = astronomic azimuth of the CO, A(CO)=A(if celestial object east of meridian) and A(CO)=360°-A(if celestial object west of meridian) for the northern hemisphere;

and

$$A(RO)=A(CO) + HCR(RO) - HCR(CO)$$

where A(RO) = Azimuth of reference object (RO)
A(CO) = Azimuth of celestial object (CO)
HCR(RO) = Horizontal circle reading on reference object
HCR(CO) = Horizontal circle reading on celestial object

OR

we may use this formula:

$$\cos A = \frac{\sin \delta - \cos Z \sin \phi}{\sin Z \cos \phi}$$

where $Z = 90° - h$

APPENDIX F

GPS Receiver Characteristics

Manufacturer	Model	SPS,PPS, No. of channels	Application / Use Envir. L, M, A	Power,Watts / Volts Hz	Wt. Lbs.	Accuracy 1 σ (m, sec)
ISTAC	2002	PPS Codeless	S / L	30 W / 12-86V dc	15	1ppm .9 conf. T. req ~ 60m
EDO Canada Ltd	JMR GeoTrak	SPS/8	S,N / L	22W	30	3ppm, .9 conf. T. req. 3 hr
Aero Service Division of Western Atlas Int'l.	MacrometerII®	PPS/6 codeless	S / L	215W / 12, 24 Vdc	120	5ppm ± 1ppm static
	Mini-Mac® 2816	PPS/6 codeless L2	S, N / L	40 W / 12-36 Vdc	40	5ppm ± 1ppm static
	Mini-Mac® 1816	SPS/2	S, N / L	40 W / 10-36 Vdc	40	5ppm ± 3ppm static
Magnavox	Wild-Magnavox Wm -101	SPS/4	S / L	25 / 12 Vdc	32	10ppm± 2ppm
Norstar Instruments Ltd.	1000	SPS/5 or SPS/7	S / L	69W / 24 Vdc	33	1-10ppm
Texas Instruments	TI-4100	PPS/4	S, N, T / L	110W / 28 Vdc	58	1ppm 1 σ supported by broadcast ephemeris
	TI-420	SPS/5	S, N, T / L	10W / 12 Vdc	10	1ppm
Trimble Navigation	4000 SX / T-SX	SPS/5	S / L	60 W / 115/230 Vac 45-66Hz 20-35 Vdc	49	10ppm± 2ppm
	4000 SL / T-S	SPS/5	S, T / L	35 W	32	10ppm± 2ppm

Note: These data are presented as submitted by representatives of each company, or taken from cited publications.

(September,1987)

Applications
N - Navigation
S - Survey T - Time

Use Environment
L - Land M - Marine
A - Aeronautical

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GLOSSARY

The majority of the definitions in this glossary were abstracted from (DoD, 1981).

Accelerometer - 1. A device that measures the rate of change of speed of an object. 2. An instrument, specially designed for carrying in aircraft or missiles, which measures the rate of change in velocity, direction, and/or altitude.

Aerial Film Speed (AFS) - A measure of speed for aerial film which replaces the formerly used aerial exposure index. It is defined as 3/2E, where E is the exposure in meter-candle-seconds at the point on the characteristic curve where the density is 0.3 above base plus fog density on black-and-white film.

Alerts - An ephemeris prepared for one or more satellites, predicting rise and set times referred to universal time coordinated, maximum angle of elevation above the observer's horizon, and azimuth from the observer. Used to identify specific satellite passes.

Altimetry - The art and science of measuring altitudes by barometric means and interpreting the results.

Altitude - Angular distance above the horizon; the arc of a vertical circle between the horizon and a point on the celestial sphere, measured upward from the horizon.

Astro Compass - An instrument used primarily to obtain true heading or true bearing by reference to celestial bodies.

Azimuth - (surveying) The horizontal direction of a line measured clockwise from a reference plane, usually the meridian. Also called **forward azimuth** to differentiate from **back azimuth.**

Backsight - 1. A sight on a previously established survey point or line. 2. (traverse) a sight on a previously established survey point, which is not the closing sight of the traverse. 3. (levelling) A reading on a rod held on a point whose elevation has been previously determined and which is not the closing sight of a level circuit; any such rod reading used to determine height of instrument prior to making a forsight. Also called **plus sight.**

Base Station (Doppler) - A previously established survey control station which is used as a reference station when positioning other stations by Doppler observations using translocation positioning techniques.

Bearing - (general) The horizontal angle at a given point measured clockwise from a specific reference datum to a second point. Also called **bearing angle**.

Bench Mark (BM) - A marked vertical control point which has been located on a relatively permanent material object, natural or artificial, and whose elevation above or below an adopted datum has been established.

Blunder- A mistake generally caused by carelessness. A blunder may be large and easily detectable, or smaller and more dangerous, or very small and indistinguishable from a random error. Blunders may sometimes be detected by repetition, or by external checks, such as closing a traverse or substituting the solution of an equation in the original.

Broadcast Ephemeris - A set of parameters broadcast by satellite from which Earth-fixed satellite positions can be computed. In particular, the parameters for the Navy Navigation Satellites (NNS) are computed for each NNS by fitting 36- to 48-hour orbital arcs to Doppler data from four tracking stations and extrapolating this arc 12 to 24 hours beyond the last data used. The length of the arc fit and the extrapolation period depend on the upper atmospheric air density. The computed parameters are injected into the satellite memory. They are transmitted along with time every two minutes.

Calibration - The act or process of determining certain specific measurements in a camera or other instrument or device by comparison with a standard, for use in correcting or compensating for errors or for purposes of record.

Clarke spheroid (ellipsoid) of 1866 - A reference ellipsoid having the following approximate dimensions. Semimajor axis - 6,378,206.4 metres; semiminor axis - 6,356,583.8 metres; and the flattening or ellipticity - 1/294.97.

Closest approach - (satellite surveying) The time and location of the satellite when it is closest to the observer/receiver antenna.

Coefficient of refraction - The ratio of the refraction angle at the point of observation to the angle at the centre of the Earth which is formed by the observer, the centre of the Earth, and point observed.

Collimation error - The angle by which the line-of-sight of an optical instrument differs from its collimation axis. Also called **error of collimation.** See collimation - level correction.

Collimation - level correction - That correction which is applied to an observed difference of elevation to correct for the error introduced by the fact that the line-of-sight through the levelling instrument is not absolutely horizontal when the bubble is centred in its vial.

Compensator - see Self-levelling level.

Constant error (zero error) - A systematic error which is the same in both magnitude and sign throughout a given series of observations, such as an index error of an instrument.

Contour - An imaginary line on the ground, all points of which are at the same elevation above or below a specified datum surface, usually mean sea level.

Control - 1. The coordinated and correlated dimensional data used in geodesy and cartography to determine the positions and elevations of points on the Earth's surface or on a cartographic representation of that surface. 2. A collective term for a system of marks or objects on the Earth or on a map or a photograph, whose positions or elevation, or both, have been or will be determined.

Coordinates - Linear or angular quantities which designate the position that a point occupies in a given reference frame or system. Also used as a general term to designate the particular kind of reference frame or system, such as plane rectangular coordinates or spherical coordinates.

Correction - A quantity, equal in absolute magnitude opposite in sign to the error, added to a calculated or observed value to obtain the true or adjusted value

Cross check lines - Data lines which cross the principal lines of development, preferably at right angles, which provide verification of, or reveal discrepancies in, the principal lines of the survey development.

Crosshairs - A set of wires or etched lines placed on a reticle held in the focal plane of a telescope. They are used as index marks for pointings of the telescope such as in a transit or level when pointings and readings must be made on an object.

Curvature correction - 1. (astronomy) A correction applied to the mean of a series of observations on a star or planet to take account of the divergence to the apparent path of the star or planet from a straight line. 2. (geodesy) The correction applied in some geodetic work to take account of the divergence of the surface of the Earth (spheroid) from a plane. In geodetic spirit levelling, the effects of curvature and atmospheric refraction are considered together, and tables have been prepared from which combined corrections can be taken.

Cyclonic circulation - The movement of winds about a centre of relatively low barometric pressure. Movement is counterclockwise in the Northern Hemisphere.

Datum - 1. Any numerical or geometrical quantity or set of such quantities which may serve as a reference or base for other quantities. 2. (geodesy) A geodetic datum is uniquely defined by five quantities. Latitude (ϕ), longitude (λ), and geoid height (N) are defined at the datum origin. The adoption of specific values for the geodetic latitude and longitude implies specific deflections of the vertical at the origin. A geodetic azimuth is often cited as a datum parameter, but the azimuth and longitude are precisely related by the Laplace condition so there is no need to define both. The other two quantites define the reference ellipsoid: the semimajor axis and flattening or the semimajor axis and semiminor axis. Also called **horizontal datum; horizontal geodetic datum.** 3. (levelling) A level surface to which elevations are referred, usually, mean sea level but may also include mean low water, mean lower low water, or an arbitrary starting elevation(s). Also called **vertical datum.**

Decca - A trade name for a radio phase-comaprison system which uses a master and slave stations to establish a hyperbolic lattice and provide accurate position-fixing.

Deflection of the vertical - The angular difference, at any place, between the upward direction of a plumb line (the vertical) and the perpendicular (the normal) to the reference spheroid. This difference seldom exceeds seconds except in mountainous terrain or great depths of the sea. Often expressed in two components, meridian and prime vertical. Also called **deflection of the plumb line, station error.**

Diapositive - (photogrammetry) A positive photograph on a transparent medium. The term is generally used to refer to a transparent positive on a glass plate used in a plotting instrument, a projector, or a comparator.

Direction - The position of one point relative to another without reference to the distance between them. Direction may be either three-dimensional or two-dimensional, the horizontal being the usual plane of the latter. Direction is usually indicated in terms of its angular distance from a reference direction.

Diurnal - Having a period of, occurring in, or related to a day.

Doppler effect - The phenomenom evidenced by the change in the observed frequency of a sound or radio wave caused by a time rate of change in the effective length of the path of travel between the source and the point of observation. Also called **Doppler shift**.

Eccentric station - A survey point over which an instrument is centred and observation made, and which is not in the same vertical line with the station which it represents and to which the observations will be reduced before being combined with observations at other stations. In general, an eccentric station is established and occupied when it is impracticable to occupy the station centre or when it becomes necessary in order to see points which are not visible from the station centre.

Elevation - Vertical distance from a datum, usually mean sea level, to a point or object on the Earth's surface, not to be confused with altitude which refers to points or objects above the Earth's surface. In geodetic formalae, elevations are heights. h is height above ellipsoid, H is height above the geoid or local datum. Occasionally the h and H may be reversed.

Ellipsoid - A surface whose plane sections (cross sections) are all ellipses or circles, or the solid enclosed by such a surface. In geodesy, ellipsoid and spheroid are used interchangeably.

Elongation - (surveying) The position of a celestial body relative to the observer's meridian, is such that the apparent azimuthal movement is at a minimum.

Epoch - 1. A particular instant for which certain data are given. 2. A given period of time during which a series of related acts or events takes place. 3. An arbitrary moment in time to which measurements of position for a body or orientation for an orbit are referred.

Error - The difference between an observed or computed value of a quantity and the ideal or true value of that quantity.

Fixed elevation - An elevation which has been adopted, either as a result of tide observations or previous adjustment of spirit levelling, and which is held at its accepted value in any subsequent adjustment.

Flight line - In air photographic reconnaissance, the prescribed ground path over which an air vehicle moves during the execution of its photo mission.

Focal length - A general term for the distance between the centre, vertex, or rear node of a lens (or the vertex of a mirror) and the point at which the image of an infinitely distant object comes into critical focus. The term must be preceded by an adjective such as "equivalent" or "calibrated" to have a precise meaning.

Foresight - 1. An observation of the distance and direction to the next instrument station. 2. (transit traverse) A point set ahead to be used for reference when resetting the transit on line or when verifying the alignment. 3. (levelling) The reading on a rod that is held at a point whose elevation is to be determined. Also called **minus sight**. See also **backsight**.

Geocentric coordinates - (terrestrial) Coordinates that define the position of a point with respect to the centre of the Earth. Geocentric coordinates can be either Cartesian (x,y,z) or spherical (geocentric latitude and longitude, and radial distance).

Geoid - The equipotential surface in the gravity field of the Earth which coincides with the undistrubed mean sea level extended continuously through the continents. The direction of gravity is perpendicular to the geoid at every point. The geoid is the surface of reference for astronomic observations and for geodetic levelling.

Gravity - 1. The force which is the resultant of the force exerted by the mass of the Earth and the centrifugal force exerted by the Earth's rotation. 2. Also the acceleration of the force defined in (1).

Greenwich hour angle (GHA) - The angle measured from the meridian of Greenwich westward to the meridian of a celestial body.

Ground swing - An error-causing condition in electronic distance measuring which is brought about by the reflection of the microwave beam from the ground or water surface. The reflected beam mixes with the direct beam at the receiving antenna, thereby changing the phase of the direct beam and causing an error in the distance measured. By varying the carrier frequency, the error becomes cyclic making possible mean instrument readings that are substantially accurate.

Gyro theodolite - A theodolite with a gyrocompass attached or built in, whereby a true azimuth reference can be established in any weather day or night, without the aid of stars, landmarks, or other visible stations. The azimuth obtained from the gyro or inertial theodolite is essentially the astronomic azimuth at the point of observation. This azimuth will differ from the corresponding geodetic azimuth by the amount of the Laplace correction.

Gyrocompass - A compass which functions by virtue of the couples generated in a rotor when the latter's axis is displaced from parallelism with that of the Earth. A gyrocompass is independent of magnetism and will automatically align itself in the celestial meridian. However, it requires a steady source of motive power and is subject to dynamic error under certain conditions. Certain aircraft compasses also use gyroscopes to gain stability, while relying basically on the magnetic meridian; these are to be distinguished from the true gyrocompass.

Gyroscope - A device consisting of a spinning rotor and associated supporting readouts which makes use of Newton's Law of Rotation to give an indication of the angular velocity of the instrument's case with respect to an inertial reference frame. This instrument is used as the basic sensor in many direction-seeking, direction-keeping, and attitude stabilization systems.

Horizon - In general, the apparent or visible juction of Earth and sky, as seen from any specific position.

Hour angle - The hour angle of a celestial body is the time elapsed since its upper transit. It is the angle between the observer's (astronomic) meridian and the declination circle of the body, measured positive westward from the meridian.

Intervalometer - A timing device for automatically operating at specified intervals certain equipment such as a camera shutter for the purpose of obtaining a desired overlap between successive photographs.

Ionosphere - The region of the atmosphere, extending from roughly 25 to 150 kilometres altitude, in which there is appreciable ionization. The presence of charged particles in this region profoundly affects the propagation of electro-magnetic radiations of long wavelengths (radio and radar waves).

Isobar - A line along which the atmospheric pressure is, or is assumed to be, the same or constant.

Kalman filtering - The recursive minimum variance estimation of an unbiased stochastic variable. An a priori estimate and covariance are linearly combined with new data to form an updated estimate and covariance.

Laplace azimuth - A geodetic azimuth derived from an astronomic azimuth by use of the Laplace equation.

Local apparent time - The apparent solar time for the meridian of the observer.

Loran C - A long-range radio navigation position fixing system using a combination of time difference of reception and phase difference of signals from two stations to provide a line of position.

Majority voting - The process of majority voting of transit Doppler data involves the accumulating of the 4.6 second Doppler counts into multiples of 6 or 7 and at the same time verifying the recorded satellite message parameters. This verification is done by comparing the messages recorded every two minutes, digit by digit, and accepting the one appearing most frequently in a particlular location.

Marker (surveying) - A definite object, such as an imprinted metal disk used to designate a survey point; sometimes refers to the entire survey monument. Mark is used with a qualifying term such as station, reference, or bench. See also **bench mark, reference mark.**

MAY'76 - A "distortion free" adjustment of first, second and third order networks in Canada on the NAD'27 datum. Adjustment was completed in 1976 and subsequent survey control has been included by transformation.

Mean sea level (MSL) - The average height of the surface of the sea for all stages of the tide, used as a reference for elevations. (Usually determined by averaging height readings observed hourly over a minimum period of 19 years). Also called **sea level datum.**

Mean square error - The quantity whose square is equal to the sum of the squares of the individual errors divided by the number of those errors.

Meridian - A north-south reference line, particularly a great circle through the geographical poles of the Earth, from which longitudes and azimuths are determined; or a plane, normal to the geoid or spheroid, defining such a line.

Met gear - Meteorological equipment for obraining dry and wet bulb temperatures and pressure. Sometimes humidity is measured in place of the wet bulb temperature.

Modulation - A variation of some characteristic of a radio wave, called the "carrier wave" in accordance with instantaneous values of another wave called the "modulating wave". These variations can be amplitude frequency, phase, or pulse.

Multi-path - See **ground swing.**

National Geodetic Vertical Datum of 1929 - Known as "sea level datum of 1929" prior to September 1973, this datum was established by contraining the combined interconnected United States and Canadian networks of first-order levelling, as it existed in 1929, to conform to mean sea level of various epochs, as determined at 21 United States and 5 Canadian long-term tidal stations distributed along the Atlantic, Gulf of Mexico, and Pacific coasts. Canada did not convert to this datum but held to values adjusted in 1927. Changes between the two datums were minimal.

North American Datum of 1927 (NAD27) - The initial point of this datum is located at Meades Ranch, Kansas. Based on the Clarke spheroid of 1866, the geodetic position of triangulation station Meades Ranch and azimuth from that station to station Waldo are as follows:

Lat. of Meades Ranch	39°13'26"686N
Long. of Meades Ranch	98°32'30"506W
Azimuth to Waldo	75°28'09"64

The geoid height at Meades Ranch is assumed to be zero. The geodetic positions of this system are derived from a readjustment of the trangulation of the entire country, in which Laplace azimuths were introduced.

North American Datum of 1983 (NAD83) - The projected datum resulting from the redefinition of the North American networks. The new adjustment of the North American networks will include a variety of geodetic data acquired since the 1927 North American datum was determined. This includes precise Geodimeter traverses, Doppler satellite positioning, astro-geodetic deflections, and gravity data.

The new North American Datum, NAD83, is a geocentric datum which uses the Geodetic Reference System, GRS 80. GRS 80 is defined by the following four parameters.

$a = 6378137$ m	The equatorial radius of the Earth
$GM = 3986005 \times 10^8\ m^3s^{-2}$	The geocentric gravitational constant of the Earth, including the atmosphere
$J_2 = 108263 \times 10^{-8}$	Coefficient of zonal potential term
$\omega = 7292115 \times 10^{-11}$ rads s^{-1}	Angular velocity of the Earth

In addition, the flattening of the reference ellipsoid is derived from the above parameters:

1/f = 298.257222...

The geocentric position of the NAD83 is derived through satellite Doppler coordinates as a subset of the North American horizontal network.

The WGS 84 coordinate system is equivalent to NAD83 inasmuch as the corrections for the Doppler Coordinate System are the same, both for NAD 83 and WGS 84, and are as follows:

* The Z coordinate correction of 4.9 m
* The scale correction of -0.6 ppm
* The longitude (E) correction of -0.5 s

NAD 83 and WGS 84 are the same for all practical applications.

Orbit - The path of a body or particle under the influence of a gravitational or other force. For example, the orbit of a celestial body or satellite is its path relative to another body around which it revolves. The term **orbit** is commonly used to designate a closed path.

Overlap - In photography, the amount by which one photograph includes the same area covered by another, customarily expressed as a percentage. The overlap between successive air photographs on a flight line is called **forward overlap** (or **forward lap**). Also called end lap. The overlap between photographs in adjacent parallel flight lines is called **side overlap** (or **side lap**).

Parallel - A circle on the surface of the Earth parallel to the plane of the Equator and connecting all points of equal latitude or a circle parallel to the aprimary great circle of a sphere of spheroid; also, a closed curve approximating such a circle. Also called **inverse parallel**.

Parameter - In general, any quantity of a problem that is not an independent variable. More specifically, the term is often used to distinguish from dependent variables quantities which may be assigned more or less arbitrary values for purposes of the problem at hand.

Pass - The period of time a satellite is within telemetry range of a data acquisition station.

Period - The interval needed to complete a cycle.

Phase angle - The phase difference of two periodically recurring phenomena of the same frequency, expressed in angular measure.

Photo-identification (surveying) - The detection, identification, and marking of ground survey stations on aerial photographs. Positive identification and location is required if survey data are to be used to control photogrammetric compilation. Also called **control-station identification.**

Pitch - The adjustable angle of helicopter blades about their lateral axis.

Plate level - A spirit level attached to the plate of a surveying instrument for levelling the graduated circle or, indirectly, making the vertical axis truly vertical.

Plumb bob - A conical device, usually of brass and suspended by a cord, by means of which a point can be projected vertically into space over relatively short distances.

Point positioning - (surveying) The process of observing Navy navigation satellites with a single Doppler survey receiver to produce the position (latitude, longitude, and height) of the receiver's antenna.

Porro prism - A prism that deviates the axis 180° and inverts the image in the plane in which the reflection takes place. It may be described as two right-angle prisms cemented together.

Precession - Change in the direction of the axis of rotation of a spinning body, such as a gyroscope, when acted upon by a torque. The direction of motion of the axis is such that it causes the direction of spin of the gyroscope to tend to coincide with that of the impressed torque.

Precise ephemeris - Coordinates and velocity of an artificial satellite, computed for uniform time intervals from data acquired from a worldwide tracking network. The ephemeris is computed from observations taken from many stations spaced worldwide and adjusted together by least-squares methods for maximum accuracy.

Precision - The degree of refinement in the performance of an operation, or the degree of perfection in the instruments and methods used when making measurements. **Precision** relates to the quality of the operation by which a result is obtained, and is distinguished from **accuracy** which relates to the quality of the result.

Prime vertical circle - The vertical circle through the east and west points of the horizon. It may be true, magnetic, compass, or grid, depending upon which east or west points are involved. Also called **prime vertical.**

Probable error - The 50 percent error interval based on the normal distribution function. See also **standard error.**

Random error - Random errors are those not classified as blunders, systematic errors, or periodic errors. They are numerous, individually small, and each is as likely to be positive as negative. Also called **accidental error, casual error.**

Reciprocal levelling - Trigonometric levelling wherein vertical angles have been observed at both ends of the line to eliminate errors.

Recompilation - The process of producing a map or chart that is essentially a new item and which replaces a previously published item. Normally, recompilation of a map or chart involves significant change to the horizontal position of features, revision of vertical values, improvement in planimetric or navigational data, or any combination of these factors.

Reference mark - A permanent supplementary mark close to a survey station to which it is related by an accurately measured distance and direction, and/or a difference in elevation.

Refraction - The change in direction of motion of a ray of radiant
energy as it passes obliquely from one medium into another in which the speed of propagation is different.

Reseau - A glass plate on which is etched an accurately ruled grid. Sometimes used as a focal-plane plate to provide a means of calibrating film distortion; used also for calibrating plotting instruments.

Residual error - The difference between any value of a quantity in a series of observations, corrected for known systematic errors, and the value of the quantity obtained from the combination or adjustment of that series. Frequently used as the difference between an observed value and the mean of all observed values of a statistically valid set. In altimeter traversing, the difference between the value of a station on the traverse corrected for datum and meteorological conditions and a known elevation. Used to adjust the elevations on the traverse linearly with distance.

Revision - The process of updating a product to reflect current information. Typically, revision of a map or chart does not required significant changes to the horizontal position of features or vertical data values; rather, improvement in planimetric data is provided. Normally, publications are revised, not recompiled.

Right ascension - The angular distance measured eastward on the Equator from the vernal equinox to the hour circle through the celestial body, from 0 to 24 hours.

Rise time - The time at which a satellite's broadcast can be picked up by a suitably equipped observer, as taken from an alert. Set time and time of closest approach are also given.

Rod level - An accessory for use with a levelling rod or a stadia rod to assure a vertical position of the rod prior to instrument reading.

Scale - The ratio or fraction between the distance on a map, chart, or photograph and the corresponding distance on the surface of the Earth.

Scale error - A systematic error in the lenghts of survey lines usually proportional to the lengths of the lines.

Sea level - The height of the surface of the sea at any time.

Self-levelling level - A level utilizing the action of gravity in its operation. A prismatic device, called a compensator, is an integral part of the instrument which, once the instrument has been roughly levelled it causes the optical system to swing into proper horizontal line-of-sight and to maintain that position during readings at a given station.

Semimajor axis - 1. (geodesy) Equatorial axis of a spheroid or ellipsoid.

Semiminor axis - One-half the shortest diameter of an ellipse.

Sensitivity - (spirit level) The accuracy and precision which a spirit level is capable of producing. Sensitivity depends on the radius of curvature of its longitudinal section; the longer the readius, the more sensitive the level. Sensitivity is rated by equating the linear length of a division between graduation marks on the level tube and its angular value at the centre of the curvature of the tube. Also called **sensibility.**

Set-up - The instrument (transit or level) place in position and levelled, ready for taking measurements, or a point where an instrument is to be or has been placed. Also called **instrument station.**

Side shot - A reading or measurement from a survey station to locate a point which is not intended to be used as a base for the extension of the survey. A side shot is usually made for the purpose of determining the position of some object which is to be shown on the map.

Sidereal time - Time based upon the rotation of the Earth relative to the vernal equinox.

Smoothed - A term used in inertial surveying to describe the adjustment of an inertial traverse between known control coordinates. Basically, this is a time dependant adjustment.

Snell's law of refraction - This law states that the sine of the angle of incidence divided by the sine of the angle of refraction equals a constant termed the index of refraction when one of the media is air. The index of refraction can also be explained as the ratio of the velocity of light in one medium to that in another.

Spherical excess - The amount by which the sum of the three angles of a triangle on a sphere exceeds 180°. In geodetic work, in the computation of triangles, the difference between spherical angles and spheroidal angles is generally neglected; spherical angles are used, and Legendre's theorem is applied to the distribution of the spherical excess. That is, approximately one-third of the spherical excess of a given spherical triangle is subtracted from each angle of the triangle.

Spheroid - (geodesy) A mathematic figure closely approaching the geoid in form and size and used as a surface of reference for geodetic surveys.

Standard error (σ) - The square root of the quantity obtained by dividing the sum of the squared errors by the number of errors minus one. The square root of the variance of the set of observations. Also called **standard deviation.**

Step wedge - An exposure from a strip of film having a series of density levels of known value.

Stereoscopic principle - The formation of a single, three-dimensional image by binocular vision of two photographic images of the same terrain taken from different exposure stations.

Striding level - A spirit level so mounted that it can be placed above and parallel with the horizontal axis of a surveying or astronomic instrument, and so supported that it can be used to measure the inclination of the horizontal axis to the plane of the horizon.

Systematic error - An error that occurs with the same sign, and often with a similar magnitude, in a number of consecutive or otherwise related observations. For example, when a base is measured with a wrongly calibrated tape, systematic errors occur. In addition, random errors will occur. Repetition does little or nothing to reduce the ill effect of systematic errors, which are a most undesirable feature of any set of observations. Much of the care in making observations is directed toward eliminating or correcting systematic errors. Also called **regular error.**

Three sigma - The 99.73 percent confidence level of a normal distribution. Three sigma is three times the value of 1 sigma. See also **standard error.**

Three-wire levelling - A method of levelling where the level's reticle has three lines. The rod is read at each of the three lines and the mean is the final rod reading.

Tidal datum - Specific tide levels which are used as surfaces of reference for depth measurements in the sea and as a base for the determination of elevation on land. Many different datums have been used, particularly for levelling operations. Also called **tidal datum plane.**

Tide gauge - A device for measuring the height of tide. It may be simply a graduated staff in a sheltered location where visual observations can be made at any desired time; or it may consist of an elaborate recording instrument making a continuous graphic record of tide height against time. Such an instrument is usually actuated by a float in a pipe communicating with the sea through a small hole which filter out shorter waves.

Tie - A survey connection from a point of known position to a point whose position is desired. A tie is made to determine the position of a supplementary point whose position is desired for mapping or reference purposes, or to close a survey on a previously determined point. To "tie-in" is to make such a connection.

Time zone - An area in all parts of which the same time is kept. In general, each zone is 15° of longitude in width, centered on a meridian whose longitude is exactly divisible by 15°.

Transformation - (surveying) The computational process of converting a position from UTM or other grid coordinates to geodetic, and vice versa, or from one datum and ellipsoid to another using datum shift contants and ellipsoid parameters. The survey position of a point is frequently given in several different grids of ellipsoid; local datum and Doppler-derived WGS 72 are common requirements.

Translocation - The determination of the relative position of two points from simultaneous satellite observations.

Trigonometric levelling - The determination of differences of elevations from observed vertical angles combined with length of lines. A type of indirect levelling.

Trilateration - A method of surveying wherein the lengths of sides of a figure are measured, usually by electronic methods, and the positions are computed from the measured lengths.

Troposphere - The lower layers of atmosphere, in which the change of temperature with height is relatively large. It is the regions where clouds form, convection is active, and mixing is continuous and more or less complete.

True north - The direction from an observer's position to the geographical North Pole. The north direction of any geographic meridian. [The term also applies to **astronomic north** to distinguish it from **magnetic north.**]

Turning point (TP) - A temporary point established between instrument set-ups to enable a survey, usually a level line, to be carried forward.

Universal time (UT) - Standard universally accepted time based on the Greenwich meridian.

Variance - The square of the standard error. Defined as the limit, as the number of observations becomes infinitely large of the sum of the squares of the residuals divided by n: the mean of the mean of the squares of errors.

Vertical angle - The angle between two lines which intersect a vertical plane. In surveying it is commonly understood that one of these lines is the horizon.

Vertical control - Elevations determined by surveying with reference to a stated datum, usually mean sea level.

Vertical photograph - An aerial photograph taken when the camera axis is as close as possible to true vertical. The resultant photograph lies approximately in a horizontal plane.

Weight - The relative value of an observation, source, or quantity when compared with other observations, sources, or quantities of the same or related quantities. The value determined by the most reliable method is assigned the greatest weight.

Wide-angle lens - A lens having a coverage angle between 75°and 100°. A lens whose focal length is equal to approximately one-half the diagonal of the format.

World Geodetic System (WGS) - A consistent set of parameters describing the size and shape of the Earth, the positions of a network of points with respect to the centre of mass of the Earth, transformations from major geodetic datums, and the potential of the Earth (usually in terms of harmonic coefficients). See also North Americal Datum of 1983.

Yaw - (air navigation) The rotation of an aircraft about its vertical axis so as to cause the aircraft's longitudinal axis to deviate from the flight line. Also called **crab**.

Zenith - That point of the celestial sphere vertically overhead.

Zenith distance - The vertical angle between the zenith and the object which is observed or defined. Zenith distance is the complement of the altitude. Also called **zenith angle**.

Zero error - See **constant error**.