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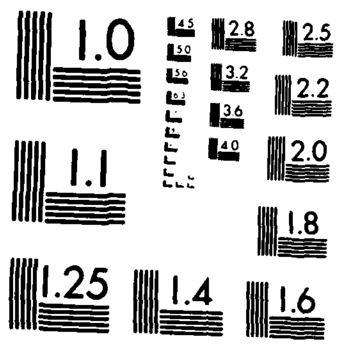

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ESTABLISHMENT OF HYDROGRAPHIC SHORE CONTROL
BY DOPPLER SATELLITE TECHNIQUES

by

David H. Minkel

June 1984

Thesis Advisors: L. D. Hothem
D. Puccini

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The methods of Doppler Satellite surveying, as applied to establishing hydrographic shore control, are presented and evaluated. Both methods, point and relative positioning, are defined procedurally with the advantages and disadvantages of each included. The field operations of two Doppler surveys (Monterey and Lake Superior) are reviewed with regard to requirements and procedures. A cost breakdown of the Lake Superior survey illustrates the high cost effectiveness of satellite techniques. The

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Establishment of Hydrographic Shore Control by Doppler Satellite Techniques

by

David Henry Minkel
Lieutenant, National Oceanic and Atmospheric Administration
B.S., Arizona State University, 1972

Submitted in partial fullfillment of the requirements for the degree of

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The methods of Doppler Satellite surveying, as applied to establishing hydrographic shore control, are presented and evaluated. Both methods, point and relative positioning, are defined procedurally with the advantages and disadvantages of each included. The field operations of two Doppler surveys (Monterey and Lake Superior) are reviewed with regard to requirements and procedures. A cost breakdown of the Lake Superior survey illustrates the high cost effectiveness of satellite techniques. The results of four Doppler data reduction programs (DOPPLR, MAGNET, GEODOP V, and MX 1502 translocation) are included and compared. Results of a special survey are included to demonstrate the high accuracy attainable by relative positioning methods. Selected data sets from both Doppler surveys were reduced using GEODOP V and are used to illustrate survey design and planning considerations. An accuracy standard for Doppler established shore control, compatible with both IHO and NOS accuracy standards, is proposed. A method for determining station elevation differences is also presented.

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I. INTRODUCTION

A critical and difficult task in conducting a hydrographic survey is determining the geographic position of each sounding. Presently, these geographic positions are determined relative to shore stations whose positions have been determined by geodetic surveying methods. The determination of the geodetic positions of hydrographic control stations frequently consumes a significant portion of the time and resources allotted to a hydrographic survey.

Before the advent of the Doppler navigational satellite system, shore stations had to be established using traditional surveying techniques. These techniques were based on operational combinations of measuring horizontal angles and distances. The particular technique used was determined by the topography of the area and the accuracy required for the survey stations.

Triangulation was for many years the favored means of establishing survey stations. This technique would yield the most accurate positions given a fixed period of time for field operations. Triangulation is a survey scheme which relies upon measuring horizontal angles between known stations while occupying an unknown station. The survey net is carried forward by forming quadrilaterals using at least two known stations and two unknown stations. The preference toward triangulation was due to the technology of the times. It was much easier to construct an instrument which could accurately measure angles rather than devising an instrument to accurately measure long distances.

Traversing is a method in which both horizontal angles and distances are measured. Normally, a traverse was run when less than maximum accuracy (first order) was required and the distance between stations was limited to a few miles. As distance measuring devices became more accurate, traversing has become a favored means of establishing highly accurate geodetic networks. Generally, traversing is less labor intensive than triangulation.

The final conventional method of establishing control is trilateration. Trilateration uses quadrilateral survey geometry like triangulation. However, it is distance between stations rather than angles that is measured[1]. Needless to say, this was not a favored means of establishing survey stations fifty years ago when 100 meter steel tapes were the instrument of distance measurement. The method has gained favor only since the advent of microwave and electro-optical (laser) distance measuring systems.

The three methods all have some commonalities. First, they all require intervisibility between survey stations, normally obtained by placing survey stations atop hills or by building observation towers over the stations. Secondly, all the instruments used have an error component which is proportional to distance. The magnitude of the error in the unknown position increases as the distance from the known station increases. Since all the methods used had a degradation of accuracy proportional to distance surveyed, standards for survey classifications were written in terms of proportional error. Hydrographic control stations are presently specified in a similar manner. Some stipulations are made as to method of establishment, but these are only for special circumstances.

---

[1]In practice, both angles and distances are measured.

With the exception of the invention and usage of Electronic Distance Measuring Instruments (EDMI), survey techniques have remained relatively similar for the last two hundred years. Equipment refinements have improved attainable accuracy, but the basic techniques have remained generally the same. The advent of EDMI did cause some change in technique and a revision of specifications, but the basic survey methods (measuring angles and distances) did not change. The situation changed in July 1967 when the Navy Navigational Satellite System (NNSS) was made available to the public. The NNSS is more commonly referred to as the TRANSIT system. This paper will refer to the NNSS as TRANSIT.

The significant difference between conventional and satellite survey techniques is that there is no requirement for intervisibility between satellite survey stations. This advantage allows station sites to be selected to optimize the survey network being established to support the hydrographic survey. The possibility of reduced costs is easily seen.

It is the purpose of this paper to evaluate the use of Doppler satellite techniques for the establishment of hydrographic shore control, and to recommend procedural as well as technical specifications. These specifications will assist the hydrographer to use Doppler in a most advantageous and efficient manner, while still acheiving the total survey accuracy required. Data was collected and processed by the author so that recommendations would be based on a working knowledge in addition to published information. The data sets are included to demonstrate specific statements or recommendations. Only one data set is included as a demonstration of the accuracy achievable with Doppler methods. Specifications quoted will be from the Hydrographic Manual of the National Ocean Service (NOS) [Ref. 1] unless noted otherwise.

II. TRANSIT SYSTEM

A. GENERAL

The Navy TRANSIT system was developed to provide a worldwide navigation system which could be used to update the inertial navigation systems aboard the Polaris submarines. The TRANSIT system was developed at Johns Hopkins University's Applied Physics Laboratory (APL) and became operational in January of 1964 [Ref. 2].

TRANSIT is a system of five operational satellites in near polar orbit. At an altitude of 1100 kilometers, the satellites complete an orbit every 107 minutes. Because the orbits are polar, satellite availability varies from once every 35 minutes to once every 100 minutes (based on five working satellites); availability being primarily a function of receiver latitude with the worst case at the equator.

Each satellite broadcasts a message which can be decoded by a ground receiver. Within this message is the position of the satellite at various times (orbital parameters) and a precise time mark. The range to the satellite is computed from the Doppler shift observed on the two ultra stable frequencies of 150 and 400 mHz broadcast by the satellites. Since the time is accurately known, the position of the satellite can be interpolated. Combining the satellite position with the range data yields the position of the receiver.

The satellite broadcasts the message beginning and ending exactly on each even minute. A time mark provides the needed time synchronization for the receiver. The message provides the smooth predicted orbit of the satellite and the time referenced deviations from the smooth orbit. This defines the satellite's position as a function of time and is referred to as the broadcast ephemeris (BE). The broadcast ephemeris is a predicted orbit based upon tracking data observed at four ground tracking stations located in Maine, Minnesota, California, and Hawaii. This tracking data, along with historical tracking data, is then used to predict the orbital elements for the next 12 hours. The BE is carried in memory on board the spacecraft and is updated every 12 hours by radio from two terrestrial computing centers (Point Mugu, California and Rosemount, Minnesota).

The precise ephemeris (PE) is actual orbital data obtained from ground tracking stations and is only available after the fact. The twenty plus worldwide tracking stations used to determine the precise ephemerides comprise the TRANET network which is maintained by the Defense Mapping Agency, *Hydrographic-Topographic Center* (DMA-HTC); the ephemerides are computed and distributed by the DMA-HTC [Ref. 3].

The receiver position is based (as previously mentioned) on the range from the satellite to the receiver. The Doppler shift observed is an accurate measure of the change in range between the satellite and the receiver for an observable time period. By intergrating with respect to time, the range to the satellite can be computed. A 30 second Doppler count consists of six or seven 4.6 second integration intervals which are averaged to yield a single range determination. Depending on the particular satellite pass, 20 to 40 range determinations can be made. The number of determinations made is dependent only on how long the satellite is visible.

Geodetic receivers have a built in clock which uses a crystal frequency standard. The standard should be stable to at least 5 X 10^{-11} parts per 100 seconds [5]. For most geodetic receivers, the clock is synchronized at the beginning of each pass via the time mark transmitted by the TRANSIT satellite. It is the receiver clock which is used for the Doppler observations and position computation. Navigational type receivers, on the other hand, do not have an internal standard, they use the timing information encoded in the satellite message. All further discussion of receiver equipment in this paper refers to geodetic receivers.

B. ERRORS

Error in the satellite derived position may come from many sources, including: an unstable frequency standard, orbital errors due to solar drag, and uncertainties in the geopotential model used to generate orbital data. Though any one source is capable of dominating, this is not usually the case. Frequency standards are quite reliable and usually introduce little error. Orbital errors, as will be discussed later can be computed or directly observed. Atmospheric refraction (tropospheric and ionospheric) would introduce a significant error (for geodetic applications) if left unmodeled.

Because the Doppler shift is due to relative motion between the satellite and receiver, receiver motion can affect the accuracy of the solution. Unknown vessel motion is the prime cause for the inaccuracy commonly associated with navigational fixes. "A reasonable rule is that 0.2 nautical mile (370 meters) of error will result from each knot of unknown ship's velocity" [Ref. 4]. Stationary receivers (such as geodetic units) will not have this error introduced into the solution.

Error due to refraction of the signal by the ionosphere is removed by comparing the wavelength stretch of both (150 and 400 MHz) broadcast carrier frequencies. The wavelength stretch is inversely proportional to the square of the transmitted frequency (first order approximation). As the path length through the ionosphere varies (with passage of the satellite), the rate of change of this wavelength stretch varies. By comparing the rate of change at both wavelengths, the error due to ionospheric refraction can be determined to a first order approximation.

Tropospheric refraction error is removed to a large part by rejecting Doppler counts recorded when the satellite is below 5 to 10 degrees above the horizon. The effect of the troposphere at 5 degrees is relatively small (approximately 26 m) when compared to the effect at the horizon (approximately 45 m) [Ref. 6]. Above 25 degrees, the error due to tropospheric refraction becomes insignificant. All Doppler reduction programs used for geodetic surveys incorporate some form of tropospheric refraction modeling to further reduce the error in the position.

C. SATELLITE DATUMS

Both satellite datums (broadcast and precise ephemerides) are nominally[2] earth centered datums as opposed to non-earth centered local datums such as NAD 1927 (North American Datum 1927). The satellite datums are referred to an earth oriented, left handed cartesian coordinate system. The Z axis is parallel to the earth's rotation axis as defined by the Conventional International Origin (CIO), positive Z is toward the North. The positive

[2]The coordinate system (NSWC 9Z-2) for precise ephemerides is known to be offset +4 meters in the Z axis [Ref. 7].

X axis passes through 0° longitude (approximately[3]) and the positive Y axis passes through 270° West longitude (Fig. 1). Orbit determinations and subsequent satellite position determinations are made independent of any reference ellipsoid. Ellipsoids are specified only to allow conversion of the X,Y,Z coordinates to geodetic coordinates (latitude, longitude, and ellipsoidal height). The equations used to compute geodetic coordinates from cartesian coordinates are shown in Figure 2.

A satellite datum is determined by the station coordinate set for the tracking stations, a geopotential model for the earth's gravity field, and four constants. These constants are: the Newtonian gravitational constant times the earth's mass, the rotation rate of earth with respect to instantaneous equinox, speed of light, and clock corrections and oscillator drift rates at the tracking stations [Ref. 9].

The BE is based on the WGS-72 (World Geodetic System 1972) geopotential (geoid) model with tracking station coordinates in the NWL-10D (Naval Weapons Lab) system. Although the resultant XYZ position is in the NWL-10D system; common practice is to compute geodetic positions (latitude, longitude, and ellipsoid height) using the WGS-72 ellipsoid constants [Ref. 10]. Many times this position is mistakenly identified as being refered to the WGS-72 datum. Because the relationship between the WGS-72 and NWL-10D datums is complex, "the only straight forward and practical procedure available is to establish a specific relationship in three dimensional coordinates (X,Y,Z) for each project" [Ref. 11]. This relationship is determined by reduction

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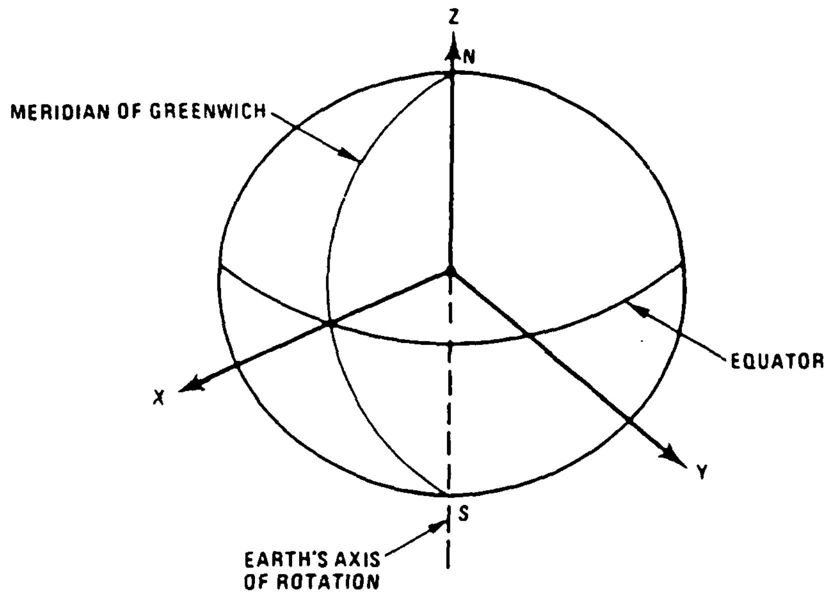
[3] Doppler longitude (East), based on the PE, needs to be increased by 0.5 to 0.8 seconds to be in agreement with the BIH zero meridian [Ref. 8].



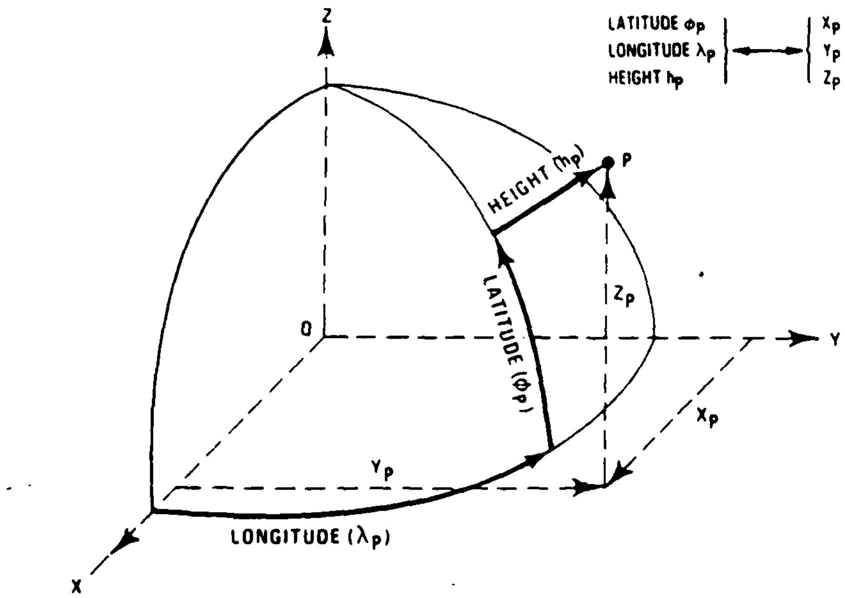


Figure 1
CARTESIAN COORDINATE SYSTEM

a = semimajor axis of ellipsoid

b = semiminor axis of ellipsoid

f = 1 - b/a

Conversion of ϕ, λ, h to x, y, z.

$$e^2 = 2f - f^2$$

$$N = a(1-e^2\sin^2\phi)^{-1/2}$$

$$x = (N + h) \cos \phi \cos \lambda$$

$$y = -(N + h) \cos \phi \sin \lambda$$

$$z = [N(1-e^2)+h] \sin \phi$$

Conversion of x, y, z to ϕ, λ, h. Formula by B. R. Bowring.

$$p = (x^2 + y^2)^{1/2}$$

$$\tan u = (z/p)(a/b)$$

$$\tan \phi = \frac{z + e'^2 b \sin^3 u}{p - e^2 a \cos^3 u}$$

$$\tan u = (1-f) \tan \phi$$

$$h = \pm [(p-a \cos u)^2 + (z-b \sin u)^2]^{1/2}$$

$$\tan \lambda = -y/x$$

The sign of h is the same as the sign of (p-a cos u).

Figure 2

Geodetic Conversion Equations

of data (one or more stations) with the PE, and transforming the resultant position to the WGS-72 datum.

The PE is based on the NSWC 9Z-2 tracking station coordinates set and the NSWC 10E-1 geopotential model. Reduction of Doppler data with the precise ephemerides yields positions in the NSWC 9Z-2 system.

Once a position has been determined in a satellite datum, a transformation can be performed to convert the coordinates to those in a desired local system. For highest accuracy, a seven parameter, three dimensional transformation is used. The seven parameters are shift of coordinate origin (3 parameters), scale change (1 parameter), and coordinate axes rotation (3 parameters).

D. FUTURE OF TRANSIT

At present, the TRANSIT system will be supported by DMA until 1992 [Ref. 12]. This date is based on time projections for deployment and testing of the operational Global Positioning System (GPS). There are presently 13 of the older Oscar series satellites and 3 NOVA series satellites in storage. Additionally, there are 8 Scout boosters reserved for launching of spacecraft as needed. There is a plan to store some of the spacecraft in orbit by boosting two satellites using a single Scout. The point of this is that the DoD is committed to supporting the TRANSIT System until 1992, and that with the spare hardware already built and paid for, there is a high probability that the system will qemain viable even in the event of budget cuts, etc. Furthermore, DoD has expressed an interest in relinquishing operation and maintenance of the system to another agency so as not to "cut off" service to the 15,000 commercial users of TRANSIT [Ref. 13].

III. DOPPLER SURVEY METHODS OF ESTABLISHING CONTROL

A. GENERAL

In most discussions of satellite positioning techniques, the term accuracy is generally not used correctly. Accuracy implies that multiple measurements of a standard have been made, that all systematic differences and blunders have been removed, and that the remaining values have been used to compute the accuracy of the measurement system. However, the quoted accuracy values are actually estimates of the true accuracy based on the statistics of the range determinations. A more appropriate term is the uncertainty of the position or range measurements. Uncertainty will be used in this paper instead of estimated accuracy. Thus "a survey with good relative uncertainties" could be read as "a survey with good estimated relative accuracies".

The two methods by which geodetic control is established via Doppler techniques are point and relative positioning. Point positioning requires only one receiver be operated; after the desired number of passes have been tracked at a station, the receiver is moved to the next station. Relative positioning requires use of two or more receivers tracking passes at two or more stations simultaneoususly. When the desired number of simultaneous passes have been recorded the receivers are moved to other stations.

B. ADVANTAGES AND DISADVANTAGES

Each will have advantages and disadvantages when compared to the other. Normally, one of the major advantages of relative positioning is the reduction in number of passes, and thus cost and time, needed to obtain the required positional uncertainty between Doppler stations.

Point Positioning (PP)

The advantage of point positioning is that only one receiver is needed, this advantage is realized in two ways:

1) Only one receiver need be bought or leased.

2) The field operations are the simplest logistically, requiring the least man power and no coordination between Doppler survey teams.

Disadvantages

1) Data reduction will take longer, due to the delay in receiving the precise ephemerides.

2) The relative uncertainties among stations may be worse and may not be easily estimated from the data reduction.

3) Data must usually be forwarded to the office for reduction since the PE is supplied weeks after the observation period in a format (magnetic tape) requiring a computing facility.

Relative Positioning (RP)

The advantages of relative positioning include:

1) The relative positional uncertainties of the survey net are usually better, and can be estimated during the data reduction.

2) Some data reduction can be performed in the field if the appropriate relative positioning firmware options are included in the receiver.

3) Data reduction is not dependent on acquisition of the precise ephemerides and may therefore be more timely.

4) Depending on the size, and required accuracy of the survey, relative positioning could be more cost efficient.

The disadvantages to relative positioning include:

1) More than one receiver is required.

2) Field operations are more complex, requiring more man power, coordination, and support equipment.

The advantages of in-field data reduction are only possible if one of the receivers has a relative position (RP) option. It should be noted, that if the RP routine fixes the orbit, the relative uncertainty cannot be directly computed [Ref. 14]. The MX-1502 allows for three orbital biases [Ref. 15], the JMR-2000 uses the semi-short arc method (up to five orbital biases) [Ref. 16], and the Motorola system uses the short arc method (6 or more orbital biases) [Ref. 17]. With these receivers the relative uncertainty is computed directly.

IV. ESTABLISHING A SURVEY NETWORK

Relative positioning scenarios are the most appropriate for the establishment of a survey network to support hydrography. The lower relative uncertainties, speed of operations, and ability to perform some data reduction in the field make this method generally superior to point positioning.

A. NUMBER OF UNITS

The first consideration to be made when planning a Doppler survey is how many receivers will be used. If the survey has more than a few stations to be occupied, at least three receivers should be used. This allows one receiver to be maintained on a high order established station in the local network. The existing geodetic control stations that are occupied are referred to as base stations. The other two units can be used to establish new stations. As the survey progresses, one of those units can be established on another base station for a few days. After the tie between base stations is made, the first unit can be moved to position a new station. This method allows the survey to continue while still making suitable ties to established control.

Two receivers on the survey allow relative positions to be computed from the data reduction. With two units, one receiver can be on a high order base station and the other unit on a new station. After a suitable number of simultaneous passes have been observed, the first unit can be moved to a new station, while the second unit is maintained on the newly established station. In this manner a Doppler traverse can be performed, eventually closing on another high order base station. A final direct tie between the two (or more) base stations would complete the survey. In either case, at least two base stations should be occupied in the survey area to insure a good tie to the existent local geodetic control. The major advantage to using at least two units is that it allows reduction of the data in the field. Those positions can be used by the field unit as the final station positions or as approximate positions until the final reduction is performed.

B. BASE STATIONS

According to the proposed FGCC specifications, if more than one Doppler station is to be established at least two (preferably three or more) base stations will be occupied [Ref. 18]. The specifications go on to state that preference should be given to stations which are tied to the National Geodetic Vertical Datum (NGVD). It is the opinion of the author that only first order horizontal network stations with ties to the NGVD should be used as base stations. The tie to the NGVD is used to determine geoidal height differences at the base stations. If two or more stations have ties to the NGVD then one can infer the geoidal slope between the two stations. This can be used to compute the elevations of the Doppler stations in the survey. If base stations with no ties to the NGVD are used, benchmarks with ties to the NGVD should also be occupied during the survey.

Other considerations to be made when selecting the base stations are the order of accuracy and age of the survey(s) which established the base stations, and the number of times the stations have been reoccupied. If the Doppler stations are to be used in conjunction with the local control during hydrographic operations, an attempt should be made to occupy, as base stations, the high order stations from which the lower order local control stations were established. As will be shown later, geodetic stations established by different surveys may not yield the same datum shift parameters. If the survey area is large, and covered by many local surveys, this procedure will allow zoning of the datum shift parameters. Zoning is simply using the datum shift of a specific area to adjust the Doppler stations in that area. This yields the best fit of the Doppler Stations to the existing local control.

The proposed revisions to the FGCC specifications stipulate that base stations should be selected which bracket the survey area. This helps to strengthen the relative position solution and the tie to the local geodetic control. They should be selected to yield a figure as close as possible to an equilateral triangle, with the survey in the center. Needless to say, the likelihood of finding such a configuration is probably poor, but this is the figure that will yield the best tie to the local geodetic control. Three base stations are required for a first order Doppler survey [Ref. 19].

If in-field relative positioning (RP) is going to be used to locate a new station which might be used in conjunction with an existing station during the hydrographic survey, the existing station should be used as the base station for the relative position solution. Since RP determines the relationship (space vector) between two stations, the unknown should be located with the same relationship as it will be used in. This removes the possibility of error which might be introduced due to the use of two different conventional surveys which may not have a direct tie between them. The same consideration should be made when establishing the station via point positioning, ie. the known station should also be occupied. The datum shift observed at the existing station can then be used to transform the Doppler position of the new station.

C. STATION SITES

Though station site selection is easier for Doppler surveys than for conventional surveys, there are still some factors which must be considered whenever any site is being investigated for possible occupation. Obstructions of the horizon, interference, security, and survey geometry all need to be considered.

One of the most critical factors at a possible site is horizon visibility. While it is advantageous to use Doppler in areas that do not require station intervisibility, obstructions affecting horizon visibility may still be a problem. At the frequencies at which the satellites broadcast, the signals require clear line of sight. The signals can be deflected, refracted, or totally obscured by obstructions between the satellite and the receiver. The sky should be clear of obstructions 7.5 degrees above the horizon. This is consistent with the cutoff angle used in most Doppler data reduction programs which, as mentioned before, is done to minimize the effect of tropospheric refraction. A 5 degree cutoff was used when selecting stations in the Monterey survey so that passes would be well clear of any obstructions.

Partial blockage, such as that caused by a lone tree, will not cause serious problems as long as no more than a few degrees in the horizontal are eclipsed by the obstruction. However a few trees could cause blockage of the satellite signals; the degree of blockage being dependent on the density of the foliage. Even if the signal is not entirely blocked, it could be refracted by the foliage. Locations near buildings (especially of metal construction) should be avoided as the signal could be reflected causing interference. This problem was observed when offshore drilling platforms were occupied; reflections from the metal deck caused an increase in the scatter of the clock error (time offset between receiver and satellite) [Ref. 20]. Placing the antenna directly on the deck caused the deck to act as the antenna's ground plane thereby reducing the amount of interference to an acceptable level.

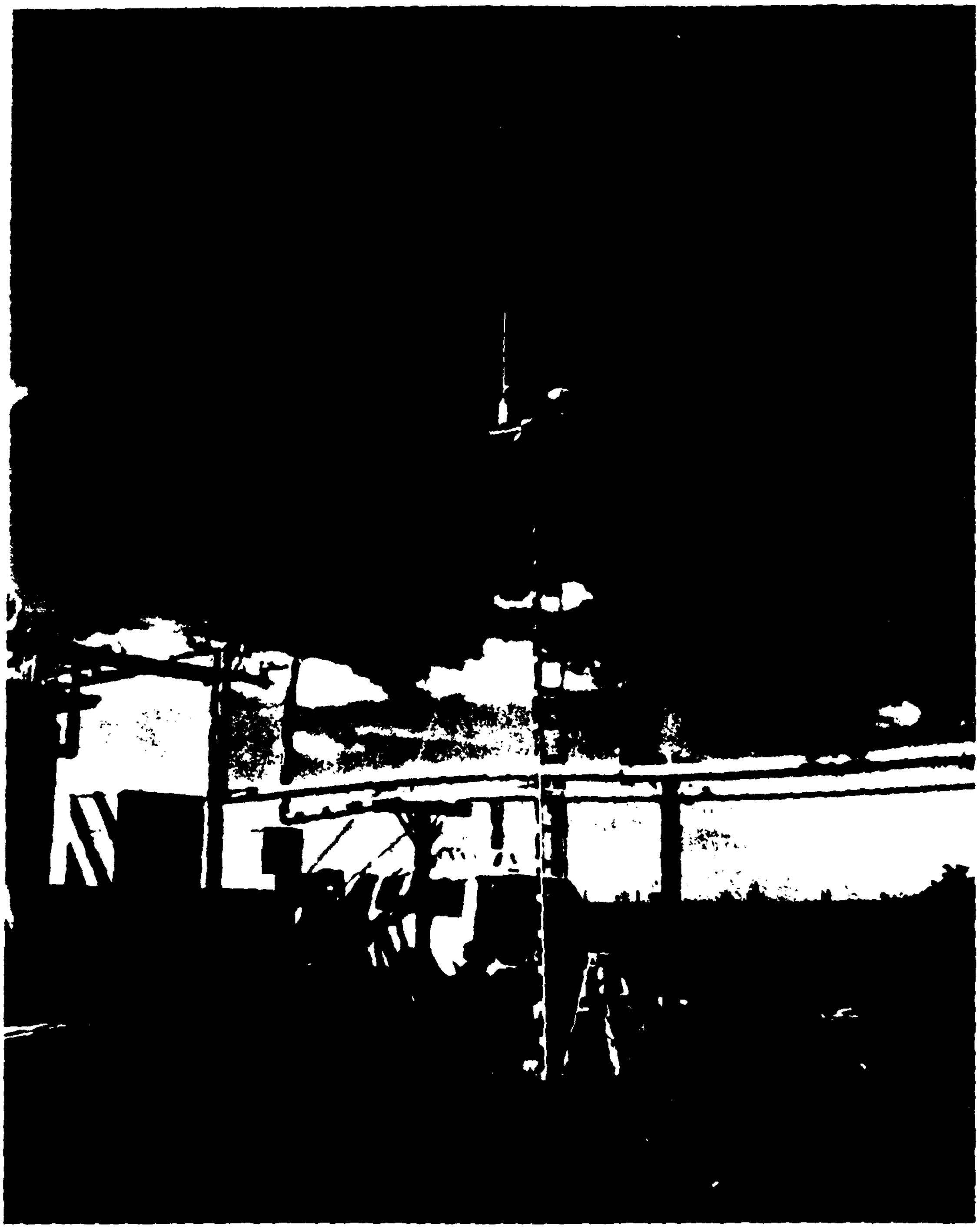
Direction of blockage is also important when selecting a site. Since the satellites are in near polar orbits, they tend to rise and set in the north and south. Blockage to the east or west of the station could cause total blockage of passes with low pass elevations. This situation would not only increase the time required on site, but it could also bias the station solution since the data set would not have a good pass balance. Pass balance refers to the desirability to have an equal number of passes, observed at maximum elevation, in each quadrant of the compass. Additionally, low level passes (8-20 degrees) are needed to adequately determine station longitude. Blockage to the north or south is not nearly so critical since data points of affected passes would only be lost during the rising and setting portions of a pass.

When a site has questionable or unacceptable visibility, but must be used, the only alternative is to elevate the antenna. This is not as large a task as with conventional survey techniques since no observations (by a person) are made on the tower. In many instances, an eight foot tripod used in place of the conventional surveyor's tripod will sufficiently improve horizon visibility. Other sites may require more elevation. During the Lake

Superior survey, two 10 foot sections of triangular antenna mast were used to elevate the antenna above obstructions (Fig. 3). The antenna was installed in a mount (tribrach) which had been bolted to a wooden 2 x 6 which projected approximately 2 feet from the tower. The tower was erected next to the station mark so that the antenna could be plumbed directly over the mark. The antenna was plumbed by placing a vertical collimator on the station mark, and bringing the center of the bolt holding the mount onto the plank into plumb. The tower guy lines were used to plumb the antenna. A 20 ft tower, of such construction, was assembled and erected in two hours, by two men, over one of the base stations in the survey. Even though a ground plane was not installed, no problem with ground reflections was noticed in the data.

Another important consideration is the possiblity of radio interference. Aeronautical radio navigation aids, television broadcast antennas, public service (fire, police) transmitters, and medium frequency radars, all broadcast near, or have harmonics near the TRANSIT frequencies. Locations near these types of transmitters should only be used with caution. Strong interference could block reception of passes and also damage the receiver. The Magnavox MX-1502 operators manual states that the instrument should not be operated within 10 meters of broadcast (TV) antennas. This distance is, presumably, to prevent damage, not insure good pass reception. Automotive ignitions and power transmissions lines radiate broadband radio frequency energy which could also effect performance of the receiver.

Site security is of prime consideration when selecting station locations. The units are designed to be as compact and portable as possible, so they are very susceptible to theft. Many sites in the Lake Superior survey were



Portable Antenna Tower

Figure 3

so remote that security was not considered to be a problem. At others, the units were chained and padlocked to nearby objects (such as navigation aids). At some stations, the unit was left inside a locked vehicle. The vehicle was located as far as possible from the antenna. Another solution which was used, was to set an eye bolt at the same time the station mark was set. The eye bolt allowed the unit to be padlocked when there were no other suitable anchors nearby. The last option which was used, is 24 hour attendance on site. This option severly limits operations since personnel are not free to perform other tasks.

Depending on the size of the the survey network and type of reduction to be performed, the orientation of the survey network might be a consideration. If the data is to be reduced using a relative positioning method, survey orientation might be important in two regards: 1) determination of orbital biases and 2) the adjustment of station positions. If the data method solves for orbit error (semi-short and short arc methods), the orientation of large surveys is important so that the magnitude and direction of the orbital errors (differences from the BE) can be most accurately determined. By comparing the difference in position shifts between two (or more) stations from pass to pass, the error in the BE can be determined. With the stations in a north-south line (same as satellite orbits), the shift differences will be at a minimum. On small surveys the north-south configuration will yield the best solutions. However, as a survey becomes larger it is necessary to change the orientation to an east-west direction. This is done to insure that stations are able to track each satellite at the same position in the oribt. Large separation in a north-south direction may preclude simultaneous observation of the satellite. Better position solutions will be reflected in the variance-covariance matrices which are input to weight the adjustment programs. The result will be an entire survey with lower relative uncertainties.

The best survey configuration is the classical quadralateral with equal side lengths. In hydrographic applications, the only situation where this survey configuration would be possible is when surveying on both sides of bays, rivers, and lakes. On long, straight shorelines, sufficient geometry can be added to the survey by occupying a base station(s) which is inland. Locating the base station(s) inland not only improves the geometry of the entire survey, and therefore, all of the station solutions, it also increases the likelihood of finding a secure, high order station to use as a base station. Examples of the effect of network configuration are given in Section XI. A..

V. DATA REDUCTION

A. GENERAL

Doppler data reduction software can be divided into two major categories: point position solutions and relative position solutions. These software categories can be further divided into programs which use the precise ephemerides (PE), those that use the broadcast ephemerides (BE), and those that use either broadcast or precise ephemerides.

Relative positioning requires that at least two stations be occupied simultaneously and that the stations track the same satellites simultaneously. The data is then used to determine the spatial relationship between the stations. Again, either the PE or BE can be used for the data reduction. There are several modes of relative positioning: translocation, rigorous translocation, semi-short arc, short arc, and simultaneous point positioning. In translocation, the assumption is made that the primary errors in the Doppler position (ephemeris error and atmospheric refraction) affect both stations equally. Therefore, the relative position is more accurate than that derived from non-simultaneous (point position) solutions. If simultaneity of data points is enforced, the translocation is termed rigorous. Translocation does not allow for corrections in the orbits. Semi-short arc allows for adjustments of up to five orbital parameters. Short arc allows for adjustments to six or more orbital parameters [Ref. 21]. Simultaneous point positioning is reduction of multiple station Doppler data (normally with the precise ephemerides) with a point position reduction program. The improved relative accuracy occurs because the stations have simultaneous observations. Even though the stations are reduced independently, any errors in the ephemerides affect all station positions identically, resulting in improved relative accuracy. The spatial relationships between the stations is not explicitly computed as in the previous methods. Much of the time, all relative positioning modes are incorrectly termed translocation by the user community.

A point position solution obtained without precise ephemeris reduction will not usually be of sufficient accuracy to meet hydrographic control specifications. When using the BE, "a horizontal positioning of 5 meters RMS can be expected with 25 satellite passes" [Ref. 22]. The best figure quoted in the literature is 3-5 meters RMS when the solution has reached convergence (approximately 40 passes). However, if this same data set is reduced using the precise ephemerides one may expect an uncertainty of 0.5 m to 1.5 m for a single point position solution. NGS' experience with program DOPPLR has shown that the uncertainty of the solution is generally at the meter to sub-meter level [Ref. 23].

The disadvantage of precise ephemeris solutions is that one must wait for the ephemerides to be computed and forwarded by DMA, which may take up to a month. Therefor thee, this type of positioning is not particularly suitable for the "on the spot" position determinations which a hydrographic field unit may wish to perform. It could be used if sufficient time exists between the control survey and arrival of the hydrographic field unit.

All of the latest geodetic Doppler receivers have the capability to do relative position solutions (though it might be an option). Generally, these solutions are in the meter to sub-meter range. MAGNAVOX claims uncertainties of ± 40 cm in latitude and longitude, and ± 1 meter in height.

This solution uncertainty is based on a data set of 16 useable passes (approximately one days data within the contiguous 48 states). FGCC test results showed differences of 12 cm in latitude, 7 cm in longitude, and 103 cm for elevation on a 42.2 km range using 29 passes for the solution [Ref. 24]. Other geodetic receivers also claim solutions of similar uncertainty levels [Ref. 25,26].

The appropriate form of data reduction for a hydrographic unit is a software system which uses the BE in a relative position solution, permiting data reduction to be done independently of the PE. These programs yield the best uncertainties within a Doppler survey while minimizing the required number of useable passes [Ref. 27]. The best internal relative uncertainties within the Doppler survey yields the best tie of the hydrographic survey to the coastline. The Doppler survey can be loosely tied to the local datum via transformation of the Doppler coordinates. If a more rigorous tie is desired, local control stations can be tied into the Doppler survey by simultaneous occupation of pre-existing and new control stations. FGCC specifications require occupation of existing geodetic stations so that a direct tie is made to the local control.

If so desired, the Doppler data observed at the existing geodetic stations can also be reduced with the precise ephemerides. This permits determination of the datum shift(s) between the Doppler coordinate system and the local geodetic coordinate system. These datum shifts can be used to analyze the local geodetic system for possible distortions. Furthermore, these datum shifts can be used to transform geodetic positions on the local datum to the satellite datum or any other datum on which Doppler observations have been made.

B. PROGRAMS USED FOR DATA REDUCTION

1. DOPPLR

Program DOPPLR is a point position solution program which uses the precise ephemerides to attain sub-meter uncertainties. DOPPLR was developed at the DMA-HTC in the early 1970's so that geodetic quality position determinations could be made from single station observations. From the beginning, DOPPLR has been based on using the PE, but the program can use the BE if so desired. Originally, the desired solution uncertainty was 1.5 m in each component, at the 90% confidence level, based on 30 to 50 useable passes. In 1977 the program was re-examined by DMA, APL, and NGS for the purpose of determining what could be done to improve the uncertainty of the solutions. The group made various improvements to the program which brought it to the current sub-meter level [Ref. 28].

The program requires: (1) the time of the beginning of the Doppler count, (2) time interval of the observation, (3) a continuous, integrated Doppler count, and (4) a refraction count. Tropospheric refraction is computed via input meteorological data and the Hopfield Model [Ref. 29].

The receiver position is computed as follows. The program computes the ranges from the satellites based on the Doppler counts. Because the orbits are held error free, each range yields a circle in space on which the receiver could be located. A block adjustment is made of all ranges, which yields the most likely intersection point of all the ranges. This intersection point is the position of the receiver.

2. GEODOP V

GEODOP V is the latest version of the GEODOP Doppler data reduction package. It was written primarily by J. Kouba and D. Boal of the Geodetic

Survey of Canada. The package has been the principal software package used to reduce Doppler Data by the Geodetic Survey of Canada since 1974. The package consists of 8 programs which are used to manipulate and process the Doppler data. Programs PREDOP and GEODOP are used to process the data, whereas the other six are utility programs used for data manipulation.

GEODOP V is a relative position software package which uses either the BE or PE. It can perform a simultaneous solution for up to 15 stations. Tropospheric refraction is modeled (4 models available) based on either input meterological data or default values. Receiver delay, frequency offset, and rate of change of frequency offset are also computed. Because the program is complex only brief descriptions of the three principal subprograms, PREDOP, MERGE, and GEODOP are included. The reader is directed to [Ref. 30] for more detail.

PREDOP is used to preprocess and edit the Doppler data collected at a single station. It also creates the Chebyshev coefficients which represent the broadcast ephemeris orbit. These coefficients are computed by the short arc method where up to six orbital biases can be computed.

MERGE is a utility program used to merge single station PREDOP output files into a single multi-station file. This file is used for processing by GEODOP.

GEODOP is the main processing program. It is used to do a pass by pass sequential adjustment of the PREDOP (or MERGE) output. GEODOP outputs geocentric cartesian and geodetic (user specified ellipsoid) coordinates for each station. A variance-covariance matrix and correlation matrix are also output. These matrices can be used to compute an estimate of the relative accuracies of the stations within the survey network.

The GEODOP system was originally written in CDC (Control Data Corp.) Fortran and designed to run on a CDC mainframe. The version used to perform the reductions in this report was obtained from Mr. Brent Archinal at Ohio State University (with the permission of Mr. Kouba). Mr. Archinal had translated the original CDC version to IBM fortran for use in his thesis work [Ref. 31]. The IBM version was installed on the IBM 370 at the Naval Postgraduate School (NPS) and the initial solutions were computed on that system. The other GEODOP solutions (Lake Superior) were computed using the NOAA UNIVAC. The UNIVAC version has been adapted from the IBM version.

3. MAGNET

Program MAGNET is a software package, developed by Magnavox, which can perform a relative position reduction of Doppler data observed by as many as 10 MX 1502 receivers. MAGNET uses the semi-short arc technique, which allows up to five degrees of freedom in the a-priori ephemeris (BE). Magnet is designed to allow three degrees of freedom: along track, across track, and in the radial direction, to allow for compensation of errors detected in the orbital coordinates.

MAGNET, similar to GEODOP V, allows the solution to "float"; where the best internal relationship of the stations is upheld. If, however, local control was occupied during the Doppler survey, the known station(s) can be constrained to the published position(s) and the remaining stations will be adjusted to yield the best result.

The preferred method would be to either: 1) determine the position differences with MAGNET, apply these differences to the base station position(s) (XYZ), then transform these coordinates to the local datum, or

2) use a network adjustment program such as NASSTI[4] or GLDSAT[5] to perform the ties to the local datum. Constraining the base stations requires that an accurate datum shift for the area be known.

Other than station and satellite coordinates, MAGNET solves for receiver frequency offset, rate of change of frequency offset, time delay from receipt of signal at the antenna to the time the Doppler count is triggered, and a tropospheric refraction correction. The tropospheric refraction correction is based on a MAGNAVOX developed model internalized within MAGNET. Weather data is not input.

MAGNET performs data reduction in 3 phases. The first is program initialization where the estimated station coordinates are input. Second, the program does 2-dimensional position computations (estimated height held fixed) modifying the Doppler data for time jitter (receiver) and first order ionospheric refraction corrections. Data is also edited if both 400 and 150 MHz channels were not tracking and when the ionopsheric refraction correction is too large. An entire pass is excluded if the maximum pass elevation was below 15 degrees. The resultant data is then stored. Third, the station solutions are then computed based on either a rigorous or simple translocation. "It is estimated that the relative accuracies of positions will not be better than 15 centimeters with any confidence in repeatability of the results " [Ref. 32].

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[4]NASSTI is an in-house NGS program used for adjustment of Doppler data.

[5]GLDSAT is used by the Geodetic Survey of Canada, to perform block adjustments of Doppler data.

Positional uncertainties obtained for two or more stations by MAGNET can be approximated by:

$$\text{SIGMA} = 150/(N(S-1))^{1/2}$$

SIGMA in centimeters

N is number of simultaneous passes

S is number of stations

This equation is probably valid for most relative positioning programs. Experience indicates the approximation is valid (slightly pessimistic) for GEODOP V solutions. One should bear in mind that this approximation is based on a station limit of ten.

4. MX-1502 Translocation Program

The MX-1502 satellite surveyor offers an on-board translocation[6] software package as an option. It allows the operator to perform a rigorous relative solution between two stations while in the field. In this form of computation the input coordinates of one station are held fixed while the other stations's coordinates are computed. The input coordinates can be either a published position or the 3-D point position computed by the receiver during the survey. The input station coordinates are compared to the coordinates obtained from a single pass solution. The difference in the two sets of coordinates is assumed to be due to error in the satellite

[6]The method used is the semi-short arc, and is therefore, not a true translocation as defined by FGCC standards. Again, common usage is to refer to any form of relative positioning as translocation.

position (based on broadcast ephemeris). This difference is then applied to the coordinate set obtained for the second station using the same pass data. This is done on a pass by pass basis for all Doppler data the two stations have in common. A data set of 17 common passes should yield a positional uncertainty of less than 1 meter. Another feature of the MX-1502 receiver is the capability to do seven-parameter BURSA-WOLF coordinate transformations. A geoidal height map (model not specified) is stored in ROM MEMORY; it is used to obtain the elevations of the stations positioned [Ref. 33].

VI. MONTEREY BAY SURVEY

To evaluate the use of Doppler positioning for establishing hydrographic control, a Doppler survey was conducted in the Monterey Bay area. The data from this survey was used to evaluate the various data reduction techniques for suitability. Additionally, the survey was designed to give a basis from which procedural specifications could be proposed. These specifications would address both the field operations and the planning required to conduct a Doppler survey. Station locations for the Doppler survey were selected based on many considerations including order of accuracy of the published position, precise elevations, network geometry, and geographic location.

A. SITE SELECTION CRITERIA

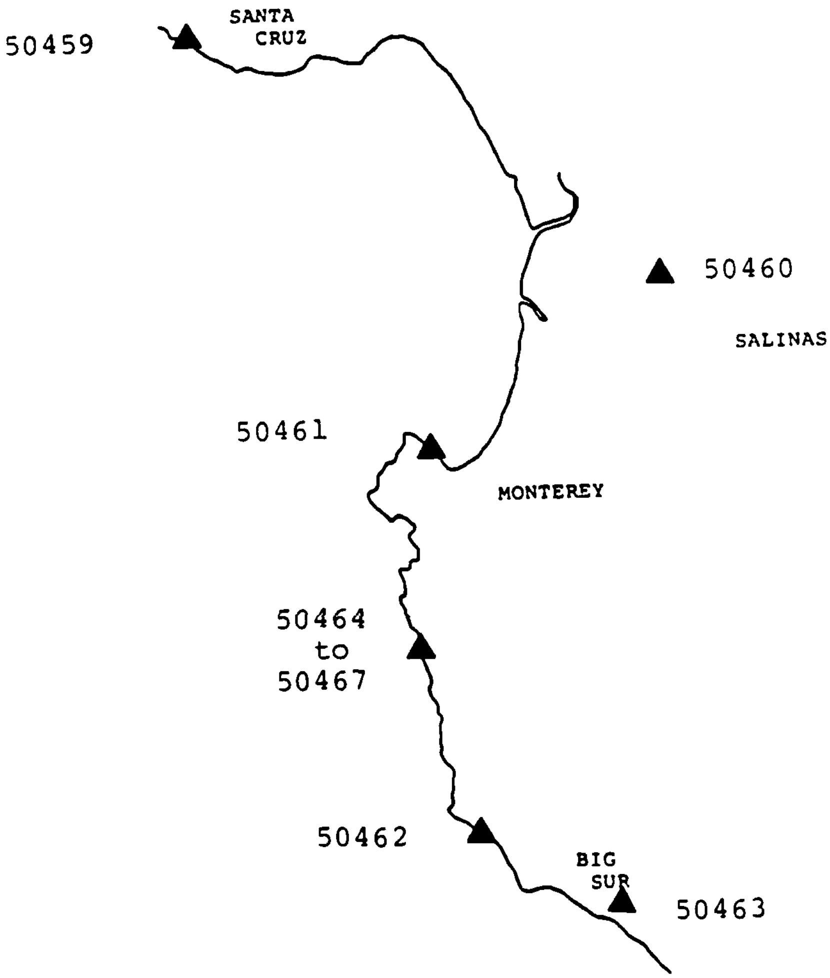
A survey station configuration was determined which would yield a data set with a large number of permutations. This would allow evaluation of the effect of network configuration and orientation on the data reduction of a relatively small Doppler survey.

All stations occupied were monumented geodetic control stations. Six stations are published stations belonging to the National Horizontal Control Network maintained by the NGS. The remaining three survey stations were established and monumented using conventional methods during the month of April 1982, prior to the Doppler survey. These stations were established as reference marks to station 50464. The decision to use established control stations was based on many factors. Primarily, the locations were already known. Use of known stations yielded a standard to which the Doppler positions could be compared. Also, an inverse computation (azimuth and distance) between the published coordinates of two stations would yield a reasonably accurate baseline distance. Secondarily, the Doppler positions obtained could be used to help NGS perform the adjustment of the network for the NAD 1983 Datum. Lastly, the positions would be easily recovered in the case of multiple occupations.

Naturally, the stations having the highest order of accuracy were given preference during the selection process. Various site factors prevented use of all but one of the first order stations considered. Four second order stations were occupied; and one third order station was used in the survey. The reference marks which were established via conventional means were at a second order station (50464). See Fig. 4 and Table 1.

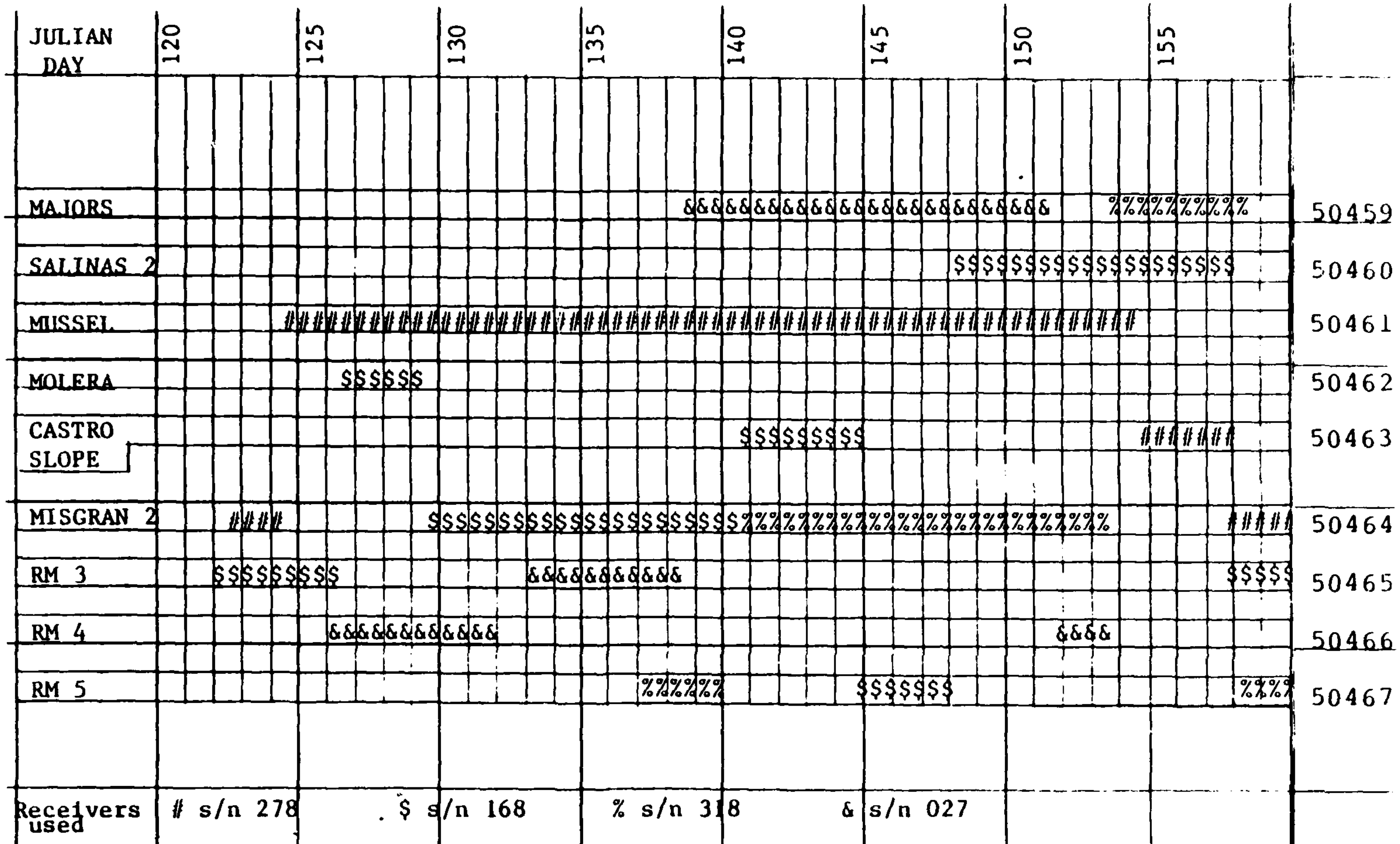
Stations with accurate elevations were also given preference during the selection process. Most of the stations occupied were at an elevation near mean sea level (MSL). One station was selected at an elevation of 826 meters. This was done to allow evaluation of the ability of Doppler to determine the elevation of occupied stations. This station was also selected since mountainous areas with limited geodetic control tend to have all the stations atop mountain peaks. Unfortunately, the station's elevation had been determined via vertical angles and not by the more accurate method of spirit leveling.

The process of station selection also considered geographic location. Since most hydrographic control stations are near the shore, and generally close to sea level, stations were selected which agreed with this general location. The near water locations were also selected because various papers



Monterey Survey Area

Figure 4



MONTEREY SURVEY OPERATIONS

TABLE 1

[Ref. 34,35] warned of possible difficulties due to multi-path interference at stations near water. Since the broadcast signals are relatively high frequency they could reflect off the water's surface. One station was selected inland to the east of the other stations occupied. This station was occupied so that the effect of network configuration could be evaluated with regard to multi-station solutions.

The preceeding described the criteria used to determine whether a station was worth the effort required to attempt recovering the station. The above criteria yielded a list of approximately twenty-five stations. Some of these stations were recovered (or an attempt made) and then evaluated on the following site considerations: accessability, visibility, security, and power. At some sites, approximately seven, it was obvious from the general location that the station would not be suitable for occupation. No attempt was made to recover these stations.

The major consideration was accessablity. The receivers required routine servicing in the form of changing data tapes, changing batteries, and checking the status of the receiver. Because the survey was to be performed by the author alone, accessability was a prime virtue. The equipment is not readily transported by one person in a single trip.

The second consideration at a location was the horizon at the station. An obstructed horizon would cause a reduction in passes tracked. A horizon clear of obstructions 5° above the horizon in all quadrants was the preferred condition. This condition was not met at all sites. One station (50464) had blockage to the east as high as 15 to 20° above the horizontal. This horizon criteria is a standard requirement for Doppler stations and is therefore not unreasonable. A data set which lacks an equal amount of passes in each quadrant may cause a bias in the height and/or longitude of the station.

The final consideration for site suitability was security. Due to the small, portable design of the receivers, they are easily stolen. The problem of security at a site was solved by one of two solutions. Either the unit was locked within one of four covered trailers leased from a local U-haul dealer, or stations were occupied on weekends when the sites could be camped on with the receivers.

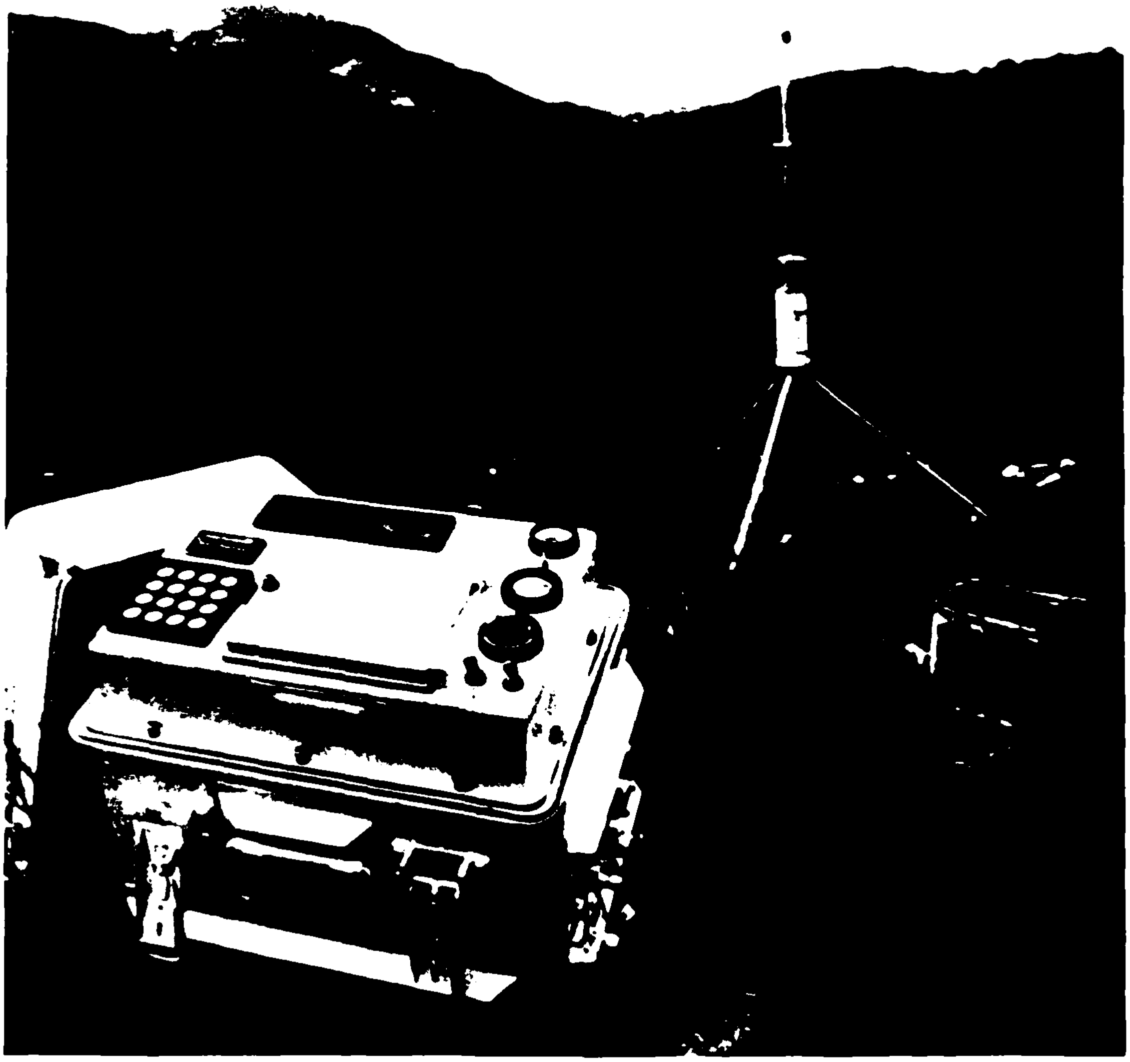
The station (50464) where the three reference marks were established was selected because it was extremely secure, and had 110 v AC power available. This site was used to verify that the receivers were in fact operating correctly.

B. SURVEY OPERATIONS

Four MAGNAVOX MX-1502 Geoceiver Satellite Surveyors were used to collect Doppler data at the various survey stations. One receiver was leased, the other three were on loan from NGS, MAGNAVOX, and the Maryland Dept. of Natural Resources. The period of the survey was from April to June 1982. All four receivers were not available for the entire survey period.

The MX-1502 is a portable, 12 v DC, geodetic Doppler receiver designed for field use (Figs. 5 & 6). Pass tracking is controlled entirely by an onboard microcomputer. The receiver is initialized and controlled via a key pad on the face of the instrument and data is displayed on a LED display window. The MX-1502 has various diagnostics for system status, and commands which allow the operator to determine the quality of the data being recorded. As a satellite pass is tracked, it is read into memory; after the computations are performed (or attempted), the pass is recorded on a cassette tape. If a position computation was possible, the solution from the computation is also recorded. The cassette is not standard in that there is a clock track recorded on the back side of the tape. Data is only recorded on one side of the tape. As the data is being recorded, it is read back and compared to memory, bit by bit, to verify that the recorded data is correct. Approximately 70 passes can be recorded on a single cassette. In Monterey, approximately 3.5 days were required to obtain 70 passes.

During operation at a site, the MX-1502 maintains two types of position based on passes tracked. The 2-D position is the position solution based only on the last satellite pass, only latitude and longitude are computed. The 2-D solution holds the height (input during initialization) fixed. This is the same form of computation that is performed in navigation type receivers. If a pass meets various criteria, such as: pass elevation, number of iterations in the 2-D computation, number of Doppler counts, and standard deviation of the residuals of the 2-D solution, it is used in the 3-D position solution. The position displayed is the culmination of all passes accepted for the 3-D solution. The update of the 3-D position is performed via a sequential adjustment using each newly accepted pass. The position computations are actually performed in X, Y, and Z; these values are converted to latitude, longitude, and height using the WGS-72 ellipsoidal parameters and stored geoidal map, and then displayed. The number of 3-D passes collected is a safe indication of how many satellite passes will be accepted for post-processing software packages. Therefore, it is a simple means of specifying the number of passes to be collected at a site. However, the criteria are specific to the MX-1502 and may not be similar in other receivers. Additionally, the



Magnavox MX 1502

Figure 5

1 Indicates voltage

2 Connects internal or external battery to meter

3 Indicates internal temperature

4 Desiccant absorbs internal moisture

5 External battery power switch

6 Operate/standby power switch

7 Fuse

8 Enters the code or data displayed

9 Numeral keys 0 thru 9 for entry of codes and data

10 Clear key

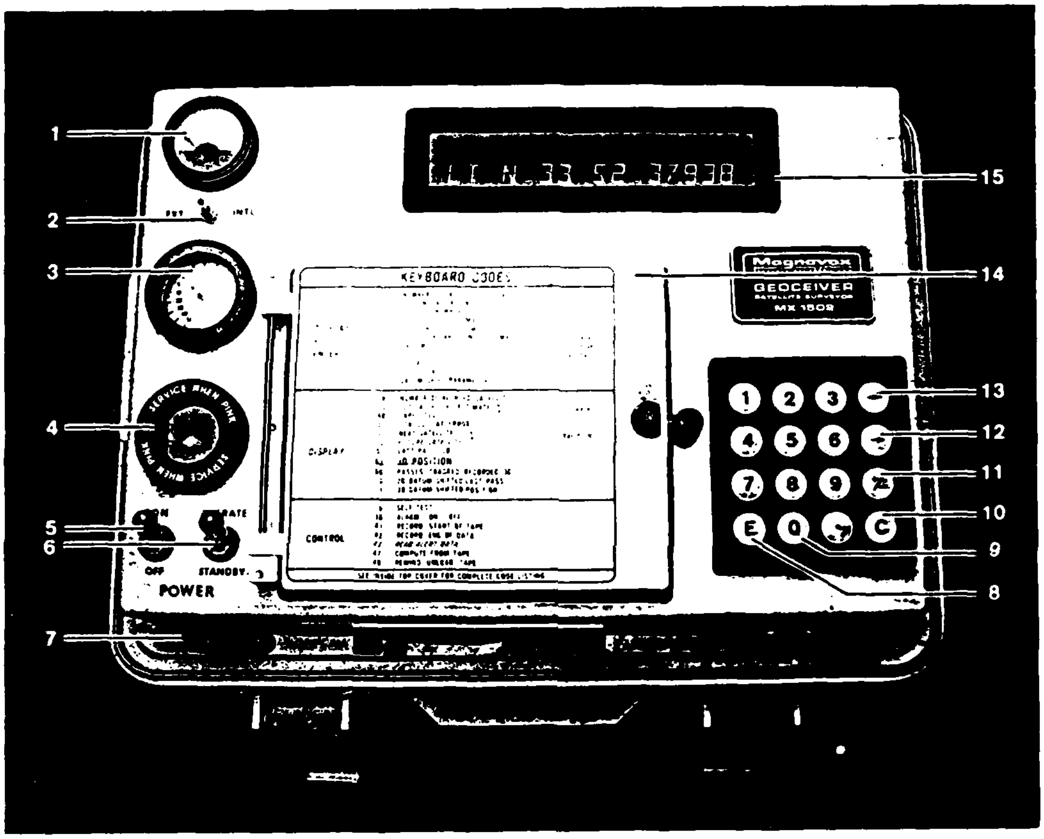
11 Change sign key

12 Space key

13 Back space key

14 Tape cassette transport

15 16 character alphanumeric display



Key to MX 1502

Figure 6

residual limit can be changed by key pad entries. If the number of 3-D passes is used to specify the number of passes to be collected, the alterable criteria should also be specified.

Use of the tripod supplied with the unit would have been cumbersome since the tripod has no provisions for leveling the head, or for horizontal movement of the antenna. By use of an adaptor, antennas were mounted on surveyor's tripods, using a conventional tribrach. This allowed for quick leveling and plumbing of the antenna over the station marks. The unit comes normally with ten and twenty meter antenna cables, a connector is also included which allows joining the two cables. At one station a sixty meter cable was used, the cable was made by NGS for use with its unit.

As stated before, the unit requires 12v. DC power for operation. During the survey, power was supplied by either using two 12 volt batteries in parallel, or by using a single 12 volt battery connected to a self-regulating battery charger (where power was available). On stations where the author camped with the units, a portable gas generator was used in conjunction with a battery charger to charge a single battery.

The unit does have two internal batteries (gel cells) which are used to maintain memory and keep the oscillator on power. When a power failure did occur no data was lost (in memory), only passes available for tracking during the power failure were lost. The unit is designed to shut down when a minimum voltage is reached.

It is important to note that this survey was conducted entirely by one person (the author) and consumed an average of 8 hrs per day. This points out the low man power requirements for surveying by satellite methods as compared to conventional methods. Units were visited and maintained on an after class basis. Batteries were usually changed every two days, and data tapes changed every three to four days.

The schedule was very tight, and did not allow for the monitoring of a pass with every visit to a station. This was not by choice, as monitoring of passes while tracking can indicate possible problems. Even so, little data was lost due to receiver failures during the survey.

VII. LAKE SUPERIOR SURVEY

The Lake Superior Doppler survey was performed to establish hydrographic control for upcoming NOS surveys scheduled for the near future. The lake area is a splendid example of an area well suited for using Doppler satellite techniques. The area is densely wooded; with the forest beginning at the water's edge in most cases. The shoreline is rugged, generally rocky, and has occasional cliff faces rising up to 150 ft above the water. Accessability from the interior to the shore is poor on both the north and south shores.

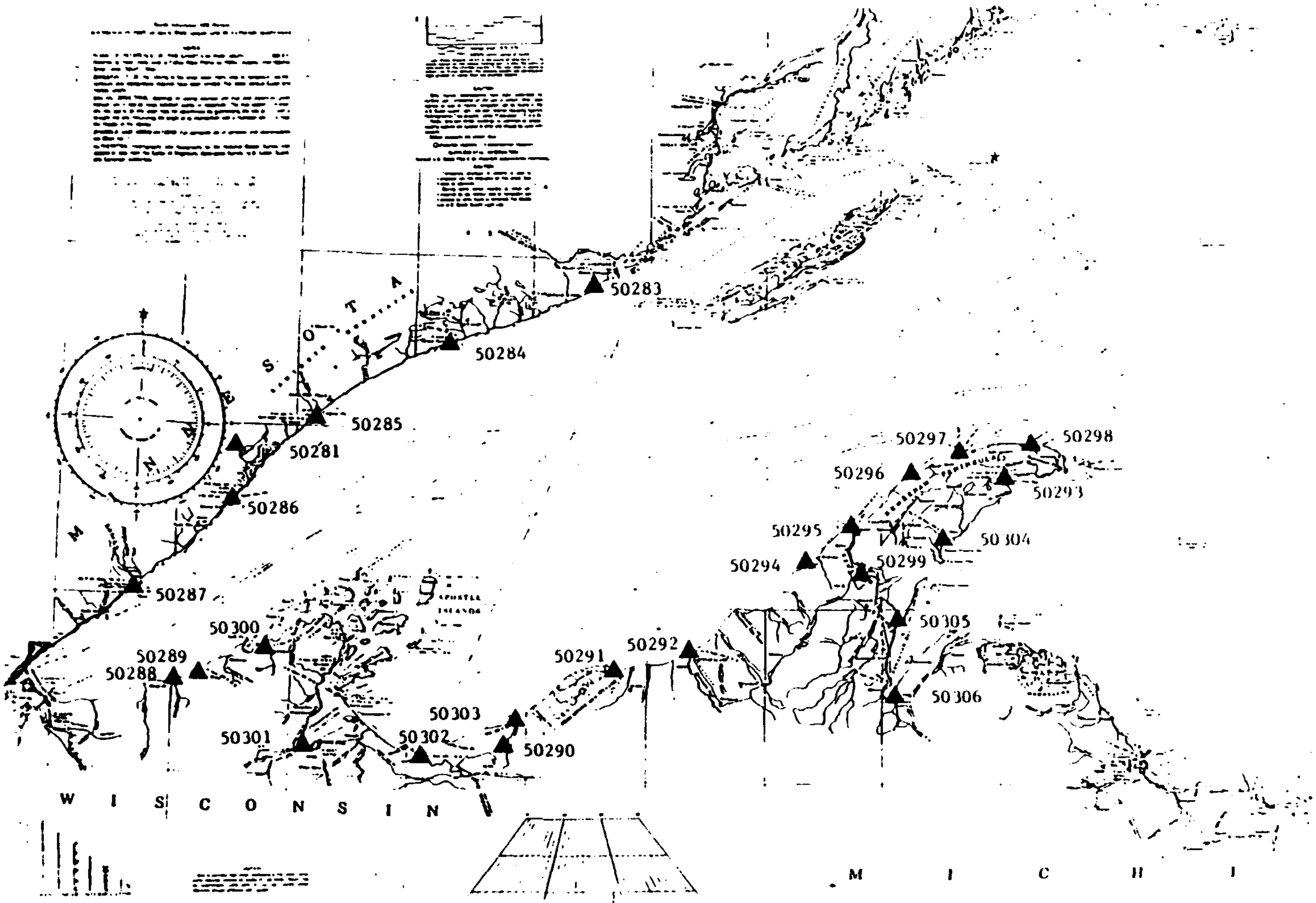
It was estimated by an advance party that to establish hydrographic control using conventional methods would require at least a full year with a crew of 8 to 12 men. It was at this point that alternate methods of establishing control were investigated by NOS personnel. In July 1982, the decision was made to establish the needed control via Doppler satellite methods. The survey was to be performed by the NOS Atlantic Marine Center (AMC), Operations Division.

In late July a planning meeting was held at the AMC. The purpose was to qeview the project area and required sites, and discuss considerations which would have to be kept in mind during the reconnaissance stage of the survey. The meeting also served as a question and answer session since most of the personnel scheduled to perform the survey had no Doppler experience. Due to the dimensions of the survey area, and the requirement for good relative uncertainty ($\pm$ 1 m) within the control network, it had been decided to use four receivers (MX-1502) simultaneously. Two of the four would be located on established first order stations, while the other two would be used to establish the needed shore control stations. In this way, all stations would be tied to one another through a few, common base stations. This common tie would allow computation of the relative uncertainties of all stations to one another. Based on a desired positional uncertainty of one meter or better, it was decided that at least 30 useable passes would be recorded at each of the stations to be established. Useable was defined for this project as a pass which had been accepted into the 3-D solution of the MX-1502 using the default residual limit values (0.25 m). The 30 pass figure was used so that the desired postional uncertainty could be obtained from a point position solution (using the PE) if need be.

The survey began in late August with the field unit (four men) conducting some of the needed reconnaissance. Some of the station marks were set at this time also. The author arrived on the evening of the 26th of August to replace one of the survey party members who had to leave, and to assist in starting the survey. The units were received and put on power on the evening of the 27th. The 28th was used to familiarize the other three men of the field party with the operation of the receivers. Since the survey party consisted of four men, four vehicles, and four receivers, when neccessary, each man could be relatively independent of the others. Independence was sometimes forced due to the size of the survey. The two fixed stations were on each end of the survey (approximately 200 miles) with each unit tended by an individual. The two mobile units were maintained by the remaining two men usually working together. These two worked together for efficiency and safety's sake. The two fixed units were set up and needed only tapes and batteries changed. Field operations commenced on the 29th of August and ran continuously until the 28th of September. Based on the schedule of the upcoming hydrographic surveys, priority was given to the north shore and the area around Duluth, Minnesota. Operations started at the most easterly station on the north shore, and progressed westerly to Duluth, then easterly along the South shore, terminating on the Keewenaw Peninsula. Stations Finland and MCM 91 were used as fixed control stations through most of the survey. With Finland in the northwest corner of the survey and MCM 91 in the southeast corner the survey area was well bracketed (Fig. 7 & Table 2).

As the work progressed to the south shore (Apostle Island area) it became necessary to start reconnaissance of more station sites. The original reconnaissance of the area had been done while the survey unit was working on another project, on a time available basis. The additional reconnaissance was performed by the three men working on the south shore. The normal daily schedule was to check the operation of the receivers in the morning, recon and/or set station marks, then return and recheck the receivers. At the latitude of the survey, it took approximately two days (an average of 44 hours) to track and record 30 useable passes. This allowed party members to perform two tasks (reconnaissance and receiver operation) at the same time, since at least every other day the units would not be moved.

During the 31 day period of the survey, Doppler positions were established on 25 survey stations, covering approximately 420 miles of shoreline. With only one exception, all stations had a minimum of 30 3-D passes before the receiver was moved.



Lake Superior Survey Area
Figure 7

JULIAN DAY	241	246	251	256	261	266	271	
FINLAND	############## ##############################							50281
HOLLOW ROCK	$$$$$$							50283
MARAIS	%%%%%%							50284
TACONITE	%%%%							50285
SILVER BAY		%%%%						50286
AGATE BAY	$$$$							50287
ANDERSON		%%%%						50288
PORT WING 2 USE		$$$$						50289
SKI		$$						50290
UNION BAY				%% %%%%				50291
CHAMPION				$$$$$$				50292
FREDA				$$$$$				50294
498 A USE				%%%%				50295
FIVE MILE PT					%%	%% %%		50296
EAGLE HARBOR					$$$$			50297
COPPER HARBOR					$$$$$$$$$$$$$$$$			50298
MCM NO 91	&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&& &&					&&&&&&&&&&		50299
CORN		%%%%						50300
BODINS			$$$$					50301
ORONTO AZ MARK			%%%%					50302
BLACK			%%%%					50303
LAC LA BELLE					%%%%%%			50293
GRAND TRAVERSE						####		50304
JACOBSVILLE						######		50305
CELOTEX						%%%%%%		50306

s/n 278 $ s/n 340 % s/n 224 & s/n 096

LAKE SUPERIOR SURVEY OPERATIONS

TABLE 2

VIII. ACCURACY STANDARDS AND SPECIFICATIONS

A. CURRENT ACCURACY STANDARDS

At present, third order, class I geodetic control is generally acceptable for use as hydrographic control [Ref. 36]. Third order, class I control is defined as having a proportional error of 1 part in 10,000. This standard is entirely relative to the station from which it is being established. The accuracy standard is specified in terms of procedural criteria which insure the desired accuracy (1:10000) will be met. The disadvantage of the current standards, is that there is no associated error ellipse or statement of error with respect to coordinate axes. The only exception to this is that the order of Doppler stations is specified in terms of the distance between stations and the relative positional uncertainty. Without the positional error of the control stations being known or included in the accuracy statement, the total error in the position of a sounding cannot be computed.

If a hydrographic chart is to be the most accurate representation of an area, all sources of error must be incorporated in any positional statement (or graphic representation). The third order class I standard does not include positional error information which might be otherwise available. If on the other hand, the standard were to be amended to include an allowable variance level associated with the station, an improved product would result. This improvement would be a more complete uncertainty value for the sounding positions depicted on the chart.

Current and pending revisions to geodetic specifications classify the order of Doppler stations based on the spacing between stations, and on the standard deviation of a single coordinate of the position solution. There are four combinations of spacing and variance which will yield a third order, class I station by point positioning (precise ephemeris) methods. If the current standard for shore control is left unchanged, the variance associated with a particular station might not be preserved or made available to the hydrographic unit.

B. PROPOSED ACCURACY STANDARDS

There are two major considerations when proposing an accuracy standard for hydrographic shore control established with satellite positioning tecniques:

1) The effects of baseline distance accuracy and azimuth accuracy between stations upon various sounding vessel positioning modes (i.e., range-range, range-azimuth, etc.) .

2) The form of data reduction to be used on the data. Will only one receiver be used, thereby forcing a point position, precise ephemeris reduction or will multiple receivers be used allowing a relative position solution ?

A simple and suitable specification would be: "all control established with Doppler satellite methods for hydrographic purposes, will be established by methods such that the station solution will have no greater than a 70 cm standard deviation in any coordinate axis if a single point position reduction (with the PE) is to be performed on the data. If a relative position reduction is to be performed, only the base stations will be occupied such that a point position reduction of the data sets would yield a 70 cm (each axis) solution New stations will be located with procedures which will yield a relative position with a 50 cm standard deviation in each coordinate axis. Doppler station spacing will conform to at least third order, class I specifications based on the reduction method which will be used. Procedures and classifications used are to conform to FGCC specifications".

The 70 cm constraint is included for two reasons:

1) Reduction of the data with the PE will yield station coordinates which can be transformed to the WGS-72 or predicted NAD 1983 datums.

2) The Doppler data could be used, via point position PE reductions, to adequately determine the relative position of one hydrographic survey to another even if the distance between the two is excessive (500 km)[7] or if the Doppler data between the surveys is not simultaneous.

Because the intrastation distance is specified so as to meet third order class I standards, the control can also be used to control aerial photography for shoreline mapping. Generally, third order horizontal control is adequate for shoreline mapping. Given enough lead time for a project, the survey team could establish both hydrographic control and photogrammetric control. If the survey was performed with multiple receivers, allowing a relative position solution, the tie between the hydrography and the photogrammetric shoreline mapping would be stronger than if the two control networks were established independently. This statement is based on the assumption that the two control networks would probably not use the same established control if other than Doppler techniques were used.

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[7] FGCC specifications limit station spacing to 500 km for relative positioning scenarios.

The same specification would also be used if fixed aids to navigation are located with Doppler satellite methods.

The specification was written to insure a station uncertainty which would be sufficient for use as hydrographic shore control based on current NOS specifications. Using the 70 cm specification, and propagating the error out to the sounding vessel, it becomes apparent that the error is not a significant contributor to the total positional error budget (Fig. 8).

Though the derivation is based on an expression that was not intended to include the positional error of the control station, it is apparent that the 70 cm standard could be incorporated into the position without a major change in other specifications. It is understood that the expression used does not include all sources of error. Expressions for error in the range due to variance in the velocity of propagation and update error are not included in the present equation[8]. Clearly, the positional error in the sounding is due mainly to the positioning system since the 3 meter sigma value is a realistic value for current ranging systems.

The proposed specification insures that the established control will meet or exceed third order, class I accuracy standards. The specification was worded so as to provide a minimum accuracy value, for every station, which could be used to evaluate the total positional error. It is a worst case statement incorporating a reasonable safety margin.

The specification is written in terms of the solution accuracy instead of number of passes so that improvements in software could reduce the number of passes needed. One very promising software improvement is an interferometric

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[8] Personal conversation with J. Wallace, NOS, Hydrographic Surveys Branch, 1983.

Using: $\sigma_p = \sqrt{2}\, \sigma_t \csc \mu$ UMBACH, pg. 4-25

Where: σ_p = drms of vessel position

σ_t = standard deviation of range

μ = angle of intersection of ranges

$$\sigma_t = (\sigma_r^2 + \sigma_s^2)^{1/2}$$

Where: σ_s = standard deviation of station position in a coordinate axis

σ_r = standard deviation of range measurement

then: $\sigma_p = \sqrt{2}\, (\sigma_r^2 + \sigma_s^2)^{1/2} \csc \mu$

To determine if incorporating the station error would have a significant effect on the ship's position, we solve for σ_r with σ_s set to .7m (the proposed specification) and set to zero. The difference will be the error in the ship's position due to the station position error.

$$\sigma_r = (1/\sqrt{2}\, \sigma_p \sin \mu)^2 - \sigma_s$$

Setting:

σ_p = 10m (.1 mm at scale of survey, assuming 1:10000)

σ_s = .7m

μ = 150° (worst case)

σ_r = 3.47m

With σ_s = 0.0m :

σ_r = 3.54m

Error Propagation

Figure 8

approach to data processing. The interferometric method is to solve for the phase difference between a single signal received at two locations. This method requires that one receiver location be known therefore it can only be used in a relative positioning mode with two receivers. Preliminary results with program SADOSA, in the interferometric mode, show baseline differences with an RMS of $\pm$ 18 cm. These measurements were made on a 39 meter baseline with two passes per solution [Ref. 37]. Further program testing , with data collected on longer baselines (up to 100 km), is not expected to show any significant difference with the preliminary results[9]. Because the interferometric mode requires a pass on each side of the observer's meridian, three or four passes may be required before an East-West pair is tracked. Therefore, this method of data reduction could reduce the required observation period, based on the specification proposed in this paper, to one third (8 hours or less) of the time presently required.

C. IHO STANDARD

This specification was also written to conform to the new (Nov 1982) shore control standards of the International Hydrographic Organization (IHO). The IHO standards state that when the shore control survey is extensive, the relative positions of control stations will not be in error by more than one half the plottable error at the scale of the survey [Ref. 38]. Using the proposed specification of 70 cm. in any coordinate axis, the relative accuracy of two Doppler stations is .99 m (1 sigma) or 1.07 m (CMAS[10]). The

[9] Personal conversation with Sz. Mihaly, Satellite Geodetic Observatory, Hungary, 1983.

[10] Circular Map Accuracy Standard

allowable relative error on a 1:5000 scale survey is 1.25 m CMAS. Therefore, the proposed specification, in the worst case (point position), will meet the IHO standards for shore control on surveys of 1:5000 or smaller (Fig. 9).

The IHO standard further specifies that satellite (or astronomic) methods should be used to establish a point of origin for the geodetic network when there is no existing network. The requirement is that the origin should have a probable error of less than 60 m. The point of origin can be established by occupying the point for the period specified required by the 70 cm. specification. The resultant point position (either PE or BE) would meet the 60 m. requirement. A PE reduction would be preferred.

It is not the purpose of this paper to recommend a new standard for all NOS hydrographic shore control. However, in the opinion of the author, the next logical standard would be a statement of acceptable positional accuracy based on the variance of the station position in any coordinate axis. The difficulty arises in that the present FGCC standards for geodetic control, do not address station error in this manner. An example of a classification system which does incorporate the error ellipse of a station into the accuracy clasification is the system used in Canada (Appendix D). With the upcoming adjustment of the North American Datum, this type of classification system would be much easier to implement since much of the distortion in the current network will be removed. Until the positional error of a geodetic position can be inferred by its order of accuracy it will not be possible to specify all hydrographic shore control by an acceptable positional error.

$\sigma_x = \sigma_y$ = standard deviation of control station position in a coordinate axis

= .7m (proposed specification)

σ_r = standard deviation of relative position

$$\sigma_r = (\sigma^2_{x1} + \sigma^2_{x2})^{\frac{1}{2}}$$

$$\sigma_r = (.98)^{\frac{1}{2}} = .99\text{m (1 sigma)}$$

$$\sigma_r \text{ CMAS}^* = 1.073\ \sigma_r$$

$$\sigma_r \text{ CMAS} = 1.062\text{m}$$

$$2\sigma_r \text{ (97\% confidence level)} = 1.98\text{m}$$

$$\sigma_p = \tfrac{1}{2}\text{(plottable error)} \times \text{(scale of survey)}$$

$$\sigma_p = \tfrac{1}{2}(.5\text{m}) \times (5000)$$

$$\sigma_p = 1.25\text{m}$$

Note: The assumption has been made that the standards are based on CMAS. No specific statement was made in the IHO standards in regard to the confidence level of the position.

* Circular Map Accuracy Standard, 90% confidence level

IHO Standard

Figure 9

IX MONTEREY SURVEY RESULTS

A. PROGRAM DOPPLR

Reduction of the Monterey Doppler data was performed by the NGS, Astronomy and Space Geodesy Section using program DOPPLR (version NGS-03). The reduction was performed as a standard production run, no special procedures or options were used. All data collected during the survey was input for reduction. The reduction was performed with ephemerides for all five satellites. It should be noted that the ephemerides for all five satellites may not always be available.

Table 3 is a summary of the datum shifts observed at the six triangulation stations which were occupied during the survey. The datum shifts shown are the origin shifts from the PE (NSWC 9Z-2) system to the local datum (NAD-27). If the PE spatial coordinates are converted to WGS-72 spatial coordinates, then differenced with the NAD-27 coordinates, the result is the datum shift from WGS-72 to NAD-27 for this area. Comparing these values to the commonly quoted shift values yields the difference in local datum shifts from the quoted mean values. This was done for station 50459 (Fig. 10) and yielded the following differences: ddx = -6 m, ddy = +6 m, and ddz = +6 m. Use of the predicted mean datum shift values in the Monterey area to perform a transformation would cause a position shift of 10 m from the local datum position. A difference of this magnitude would cause significant errors if a transformed position (to NAD-27) were used with already existent geodetic control to position a hydrographic survey. If point positioning techniques are to be used to establish hydrographic shore control, the datum shift for the survey area must be determined by occupation of existent geodetic control.

Table 4 is a summary of the point position reduction for the Monterey survey. Observations refers to the number of 30 second Doppler counts[11]. The geocentric coordinates shown are derived from the PE and are nominally earth centered. The height shown is the ellipsoidal height, not elevation of the station. The high rejection rate observed on station 50462 is due to an error in the data collection. Some data (8 passes) observed at 50463 were erroneously marked as from 50462. When the data was processed, these passes were rejected due to the position misclosure. Removal of this data would bring the rejection rate to 4%.

Station 50466 and 50467 both have two solutions summarized due to different occupations. In both cases, the antennas were not re-established close enough to the original antenna height to allow reduction as a single station. FGCC specifications require the antenna height be re-established within ± 0.005 m of the original antenna height. All reduction programs reduce the data at the phase center of the antenna, then correct the final position to the survey mark. This means the data set must be subdivided if there are multiple antenna heights otherwise the solution will have a high RMS.

Table 5 is a summary of the station positions in the local datum. These are the positions as determined from conventional methods. Stations 50465,

[11] The observations RMS is the root mean square of the ranges which are computed from the 30 second Doppler counts. This value can be used as an indicator of the quality of the data. A value of .30 m or higher would indicate a poor data set.

STATION NUMBER	SOL CODE	CITY OF TOWN	STATION LOCATION COUNTRY	STATE	LATITUDE(N) D M S	LONGITUDE(W) D M S	DATUM SHIFT (LOCAL DX (M)	NSWC 9Z 2) DY (M)	DZ (M)	SURFACE SHIFT N(X) (M)	E(Y) (M)
50459	1003	MAJORS	US	CA	36 58 47	122 9 11	35.90	-162.74	-184.90	8.82	117.00
50460	1003	SALINAS	US	CA	36 45 32	121 40 3	35.34	-162.15	-184.70	7.93	115.21
50461	1003	MONTEREY	US	CA	36 37 18	121 54 12	35.85	-162.52	-184.58	7.93	116.33
50462	1003	BIG SUR	US	CA	36 16 54	121 51 40	36.36	163.02	-184.82	6.67	116.94
50463	1003	BIG SUR	US	CA	36 13 21	121 43 48	36.99	-163.06	184.38	6.88	117.21
50464	1003	CARMEL	US	CA	36 26 23	121 55 17	35.99	-163.13	-184.90	6.93	116.80
					MEAN DATUM SHIFT		36.07	-162.77	-184.71		
					STANDARD DEVIATION OF MEAN		.56	.38	.20		

Summary of Datum Shifts

Table 3

From MEADE:

$$X_w = X_p - (0.827\ X + 1.26\ Y)\ 10^{-6}$$

$$Y_w = Y_p - (0.827\ Y - 1.26\ X)\ 10^{-6}$$

$$Z_w = Z_p - (0.827\ Z)\ 10^{-6}$$

Where: w denotes WGS-72 and p denotes NSWC 9Z-2 (PE)
For station 50459:

$$\begin{aligned} X_p &= -2714956.98 \qquad & X_w &= -2714949.29 \\ Y_p &= -4318894.90 & Y_w &= -4318894.75 \\ Z_p &= 3815579.15 & Z_w &= 3815576.00 \end{aligned}$$

Differencing the published cartesian coordinates with the above WGS-72 coods.:

$$\begin{aligned} X_1 - X_w &= 28.21 = dx \\ Y_1 - Y_w &= -162.89 = dy \\ Z_1 - Z_w &= -181.75 = dz \end{aligned}$$

These resultant values are the datum shift from WGS-72 to NAD 27, differencing these with the published (dx_p = 22m, dy_p = -157m, dz_p = -176m) *datum shift* values yields:

$$\begin{aligned} dx_p - dx &= -\ 6.21 = ddx \\ dy_p - dy &= 5.89 = ddy \\ dz_p - dz &= 5.75 = ddz \end{aligned}$$

$(ddx^2 + ddy^2 + ddz^2)^{1/2}$ = 10 m (distance due to error in datum shift values)

Note: the "published values" are the origin shifts needed to make the Clarke 1866 ellipsoid (NAD 27) coincident with the WGS-72 ellipsoid, the source of these values was Appendix A of the MX-1502 Operator's Manual.

Computation and Comparison of Datum Shift
Figure 10

50466, and 50467 are not shown since they were not established (published) horizontal control stations.

Tables 6 and 7 show the differences between the transformed Doppler positions and the published positions. The differences can be used as an

STAT NUM	SOL CODE	SAT #	PERIOD (DAY NO) FM TO YR	PASSES USED/IN	OBSERVATIONS USED/IN	OBS ZR	RMS (M)	GEOCENTRIC COORDINATES X (M)	Y (M)	Z (M)	GEODETIC COORDINATES (WGS-66) LAT(N) D	M	S	LONG(E) D	M	S	HEIGHT (M)
50459	1003	**	139-159/82	277/ 290	5718/ 6081	6	.11	-2714956.98	-4318894.90	3015579.15	36	58	46.3928	237	50	44.1022	9.75
50460	1003	**	148-157/82	133/ 133	2876/ 2909	1	.11	-2685983.51	-4354264.00	3796008.78	36	45	32.2346	238	19	51.9881	14.70
50461	1003	**	125-154/82	460/ 465	9707/10145	4	.10	-2708658.78	-4350875.43	3783759.26	36	37	17.8938	238	5	43.6904	35.98
50462	1003	**	126-140/82	53/ 63	1119/ 1354	17	.12	-2717302.91	-4371908.67	3753427.72	36	16	54.1295	238	8	15.0735	19.08
50463	1003	**	140-157/82	107/ 109	2189/ 2392	8	.11	2709667.43	4381974.13	3740607.41	36	13	21.0908	238	16	7.4222	784.91
50464	1003	**	122-153/82	418/ 420	7782/ 8340	7	.14	2716387.91	-4360228.32	3767551.03	36	26	23.1595	238	4	38.7077	21.37
50465	1003	**	122-159/82	149/ 149	2757/ 2952	7	.13	-2716391.67	4360235.74	3767541.10	36	26	22.7406	238	4	38.7371	20.60
50466	1003	**	137-139/82	44/ 45	894/ 949	6	.14	-2716386.61	-4360254.94	3767523.08	36	26	22.0079	238	4	39.3172	20.34
50466	2003	**	144-159/82	67/ 67	1292/ 1349	4	.12	-2716386.55	-4360255.41	3767523.79	36	26	22.0193	238	4	39.3292	19.63
50467	1003	**	126-131/82	91/ 91	1718/ 1792	4	.13	-2716390.16	4360247.43	3767527.59	36	26	22.2122	238	4	39.0367	21.28
50467	2003	**	152-153/82	26/ 26	459/ 480	4	.12	-2716389.57	-4360248.13	3767527.51	36	26	22.2047	238	4	39.0717	21.10

REMARKS: WGS-66 ELLIPSOID CONSTANTS: A= 6378145.0 M., F= 1/298.25
**- SOLUTION WITH MORE THAN ONE SATELLITE
DOPPLER COORDINATES DERIVED WITH PROGRAM 'DOPPLR' USING THE PRECISE EPHEMERIDES.
PROGRAM VERSION: NGS-03 (8 DEGREE CUTOFF AND 10% TROPO SCALE BIAS).
COORDINATES ARE REFERENCED TO STATION MARK.

Summary of Point Positioning Results (Monterey)

Table 4

STATION NUMBER	SOL CODE	GEODETIC COORDINATES LATITUDE DIR D M S	LONGITUDE DIR D M S	ELEVATION (MSL) (M)	GEOID UNDULATION (M)	ELLIPSOID HEIGHT (M)	CARTESIAN COORDINATES X (M)	Y (M)	Z (M)
50459	1003	N 36 58 46.6790	W 122 9 11.1670	31.93	-33.8	-1.87	-2714921.08	4319057.44	3815394.25
50460	1003	N 36 45 32.4920	W 121 40 3.3670	55.7	-32.4	23.3	-2685948.17	-4354426.15	3795824.08
50461	1003	N 36 37 18.1510	W 121 54 11.6280	6.1	-33.8	-27.68	-2708622.93	-4351037.95	3783574.68
50462	1003	N 36 16 54.3460	W 121 51 40.2410	23.4	-34.2	-10.76	-2717266.55	-4372071.69	3753242.90
50463	1003	N 36 13 21.3140	W 121 43 47.8850	828.0	34.6	793.44	-2709630.44	-4302137.19	3748423.03
50464	1003	N 36 26 23.3844	W 121 55 16.6026	21.25	-34.2	-12.95	-2716351.92	-4360391.45	3767366.13

REMARKS:
COORDS ARE REFERENCED TO STATION MARK.

Summary of Local Datum Coordinates (Monterey)

Table 5

indicator of the quality of the local geodetic network. Doppler derived positions can be expected to have an internal precision of approximately 30 cm (1 sigma) when observations were performed simultaneously and sufficient passes (30 or more) were observed [Ref. 39]. If one observes high variation in the coordinate difference values, the local network lacks internal precision. Changes of sign with the associated magnitudes seen in Tables 6 and 7 indicate the local geodetic network in the Monterey area lacks internal precision. This lack of precision is due, in part, to the stations not having ties to one another. This lack of consistency in the coordinate differences would not generally be found in geodetic stations which had all been established with the same survey.

Table 8 can be used to determine if there is a scale difference between the Doppler coordinate system and the local geodetic system. By computing the baseline differences in parts per million for each baseline, and meaning these differences, one can detect scale difference. The standard deviation of the mean should also be computed to determine if the mean is realistic. Doppler (NSWC 9Z-2) and NAD 27 have a scale factor of about -0.5 ± 0.04 ppm [Ref. 40]. A scale factor this small would produce negligible differences on a survey as small as the Monterey survey.

B. PROGRAM MAGNET

Reduction of the Monterey Doppler data with program MAGNET (version HP 80256) was performed by Mr. Robert Skeans, MAGNAVOX Corporation. Therefore, the procedures and options used are not as well known to the author as those used for the other reduction programs. The following discussion is based on the program output and program documentation supplied by Mr. Skeans.

STATION NUMBER	SOL CODE	STATION LOCATION LATITUDE DIR	D	M	S	LONGITUDE DIR	D	M	S	COORDINATE DIFFERENCES (LOCAL - DOPPLER) LATITUDE (SEC)	LATITUDE (CM)	LONGITUDE (SEC)	LONGITUDE (CM)	HEIGHT (CM)
50459	1003	N	36	58	47	W	122	9	11	-.0010	-3	.0070	17	-6
50460	1003	N	36	45	32	W	121	40	3	.0081	25	.0246	61	-10
50461	1003	N	36	37	18	W	121	54	12	.0104	32	-.0006	-1	0
50462	1003	N	36	16	54	W	121	51	40	.0011	3	-.0284	-71	0
50463	1003	N	36	13	21	W	121	43	48	.0183	56	-.0508	-127	0
50464	1003	N	36	26	23	W	121	55	17	-.0067	-21	-.0182	45	17
		ARITHMETIC MEAN (N = 6)								.0050	15	.0134	33	0
		STANDARD DEVIATION (RMS)								.0090	28	.0257	64	9
		SPREAD								.0250	77	.0753	188	27
		MAXIMUM								.0183	56	.0246	61	17
		MINIMUM								-.0067	21	.0508	127	-10

REMARKS: DX= 35.74, DY=-162.67, DZ=-184.84, RX= .00, RY= .00, RZ= .00, K= .00PPM
DOPPLER COORDINATES. DERIVED WITH PROGRAM 'DOPPLR',
VERSION - NGS-03. SHIFT DETERMINED FROM
MEAN SHIFT FOR STATIONS 50459, 50460, AND 50464.

Comparison of Transformed Doppler and
Local Datum Coordinates (mean)

Table 6

STATION NUMBER	SOL CODE	STATION LOCATION LATITUDE DIR	D	M	S	LONGITUDE DIR	D	M	S	COORDINATE DIFFERENCES (LOCAL - DOPPLER) LATITUDE (SEC)	LATITUDE (CM)	LONGITUDE (SEC)	LONGITUDE (CM)	HEIGHT (CM)
50459	1003	N	36	58	47	W	122	9	11	.0000	0	.0000	0	-1
50460	1003	N	36	45	32	W	121	40	3	.0092	28	.0315	78	-5
50461	1003	N	36	37	18	W	121	54	12	.0115	35	.0064	16	6
50462	1003	N	36	16	54	W	121	51	40	.0022	7	-.0215	-54	5
50463	1003	N	36	13	21	W	121	43	48	.0194	60	-.0439	-110	6
50464	1003	N	36	26	23	W	121	55	17	-.0056	-17	-.0112	-28	23
		ARITHMETIC MEAN (N = 6)								.0061	19	-.0065	-16	6
		STANDARD DEVIATION (RMS)								.0090	28	.0257	64	9
		SPREAD								.0250	77	.0754	188	27
		MAXIMUM								.0194	60	.0315	78	23
		MINIMUM								-.0056	-17	-.0439	-110	-5

REMARKS: DX= 35.90, DY=-162.74, DZ=-184.90, RX= .00, RY= .00, RZ= .00, K= .00PPM
DOPPLER COORDINATES: DERIVED WITH PROGRAM DOPPLR ,
VERSION - NGS-03. SHIFT DETERMINED AT 50459.

Comparison of Transformed Doppler
and Local Datum Coordinates (50459)

Table 7

STATIONS FROM	TO		BASELINE VECTOR DX (M)	DY (M)	DZ (M)	B (M)	DIFFERENCES (DOPPLER MINUS OTHER) DDX (M)	DDY (M)	DDZ (M)	DB (M)
50459	50460	DOPPLER	-28973.35	35369.32	19570.19	49733.65				
1003	1003	OTHER	-28972.79	35368.74	19569.99	49732.83	-.56	.58	.20	.82
50459	50461	DOPPLER	-6298.05	31980.78	31819.69	45551.38				
1003	1003	OTHER	-6298.00	31980.56	31819.37	45551.00	-.05	.22	.32	.39
50459	50462	DOPPLER	2345.92	53013.77	62151.46	81723.73				
1003	1003	OTHER	2345.47	53014.06	62151.37	81723.84	.45	-.29	.09	-.11
50459	50463	DOPPLER	-5289.44	63079.44	66971.60	92153.07				
1003	1003	OTHER	-5290.53	63079.76	66971.08	92152.98	1.09	-.32	.52	.10
50459	50464	DOPPLER	1431.46	41334.29	48027.39	63381.41				
1003	1003	OTHER	1431.38	41334.68	48027.39	63381.66	.08	-.40	.00	-.26
50460	50461	DOPPLER	22675.30	-3388.54	12249.50	25994.26				
1003	1003	OTHER	22674.79	-3388.18	12249.38	25993.71	.51	-.36	.12	.55
50460	50462	DOPPLER	31319.27	17644.45	42581.27	55726.01				
1003	1003	OTHER	31318.26	17645.32	42581.39	55725.80	1.01	-.87	-.11	.21
50460	50463	DOPPLER	23683.91	27710.11	47401.41	59796.92				
1003	1003	OTHER	23682.26	27711.01	47401.09	59796.43	1.65	-.90	.32	.49
50460	50464	DOPPLER	30404.81	5964.96	28457.20	42069.53				
1003	1003	OTHER	30404.17	5965.94	28457.40	42069.34	.64	-.98	-.20	.19
50461	50462	DOPPLER	8643.98	21032.99	30331.77	37909.39				
1003	1003	OTHER	8643.47	21033.50	30332.00	37909.74	.50	-.51	-.23	-.35
50461	50463	DOPPLER	1008.62	31098.66	35151.91	46944.65				
1003	1003	OTHER	1007.48	31099.20	35151.71	46944.84	1.14	-.54	.20	-.18
50461	50464	DOPPLER	7729.52	9353.51	16207.70	20246.56				
1003	1003	OTHER	7729.38	9354.12	16208.02	20247.05	.13	-.61	-.32	-.49
50462	50463	DOPPLER	-7635.36	10065.66	4820.14	13522.20				
1003	1003	OTHER	-7636.00	10065.69	4819.71	13522.43	.64	-.03	.43	-.23
50462	50464	DOPPLER	-914.46	-11679.49	-14124.07	18350.37				
1003	1003	OTHER	-914.09	-11679.38	-14123.98	18350.21	-.37	-.11	-.09	.15
50463	50464	DOPPLER	6720.90	-21745.15	-18944.21	29612.58				
1003	1003	OTHER	6721.91	-21745.07	-18943.69	29612.42	-1.01	-.08	-.52	.16

	DDX	DDY	DDZ	DB
ARITHMETIC MEAN	.39	-.35	.05	.09
STANDARD DEVIATION (RMS)	.71	.43	.29	.37
N = 15				
SPREAD	2.66	1.56	1.04	1.31
MAXIMUM	1.65	.58	.52	.82
MINIMUM	-1.01	-.98	-.52	-.49

Comparison of Baseline Vectors

Table 8

To maximize the amount of data used in the reduction, a pass only had to be tracked at two stations to be accepted into the solution. In areas where all stations have good horizon visibility this would have little effect on the size of the final data set. In the Monterey survey, a noticeable portion of passes could have been excluded if the requirement had been that three stations must track a pass. This is due to two factors: 1) only four receivers were used in the survey and many times one was being transported 2) stations 50464, 50465, 50466, and 50467 had poor visibility to the east and would not "see" low level passes in that direction.

The maximum RMS value for the errors of a posiiton fix was set at 17 cm. The position fix is the range from the satellite based on the 30 second Doppler count. The RMS of the six or seven 4.6 second Doppler determinations could not exceed 17 cm without the 30 second Doppler count being rejected.

The frequency drift of the oscillators was not computed in the reduction. This condition was imposed because more than one receiver had been used on some stations. To accurately solve for receiver characteristics such as frequency drift and receiver time delay, station data sets must be subdivided into single receiver data sets. This would require that the subsets be processed as separate stations.

MAGNET adjusts three parameters of the satellites' orbits. These orbital biases were constrained to 24 m, 4 m, and 9 m; for along track, height and cross track, respectively.

Pass cutoff was set at 5° (above the horizon); no Doppler data below a 5° elevation is used. Furthermore, a pass was not used if the maximum elevation did not reach 14.5° [Ref. 41]. All other reduction programs with which the author is familiar use a 7.5° cutoff. As mentioned before, this cutoff value is specified to help minimize error due to tropospheric refraction.

The one sigma estimates of latitude, longitude, and antenna height are shown for each station in Table 9. These values are the uncertainties of the station positions relative to the other stations. The estimated standard deviation of unit weight is not output, so the validity of these estimated accuracies is not strictly known. Based on experience with other reductions these values do seem realistic.

Baseline lengths determined by this reduction are compared with baselines determined by the other reduction programs in Table 22, section XIV.

The results of this reduction may not be optimal. The major change in the reduction which should improve solution accuracy would be division of data sets into subsets of a single occupation. The improved uncertainties would be due to each subset having data from only one receiver obtained at a single antenna height. The errors induced by combining all Doppler data observed at a single station may have been somewhat reduced by the large size of the entire data set. Division of the data sets would have forced two reductions to be performed due to the station limit (10) of MAGNET.

C. MX-1502 TRANSLOCATION PROGRAM

To evaluate the accuracy and ease of performing field computed positions three data sets were reduced using the MX-1502 Field Translocation option. The processing was not performed while the unit was on site tracking. Instead, the computations were performed at a later date, after the data collection phase had been completed.

In brief, the computation procedure is to input, into the MX 1502, the final 3-D positions, determined in the field via point positioning, of both the remote and control stations. The first acceptable seventeen passes are

STATION	LATITUDE	LONGITUDE	HEIGHT	X	Y	Z
50459	36 58 46.292	122 09 15.908	2.56	-2714959.06	-4318897.73	3815578.78
50460	36 45 32.139	121 40 08.020	27.34	-2685985.60	-4354267.01	3796008.74
50461	36 37 17.792	121 54 16.315	-23.37	-2708660.88	-4350878.54	3783759.03
50462	35 16 54.034	121 51 44.918	-6.30	-2717304.71	-4371911.99	3753427.72
50463	36 13 20.997	121 43 52.579	797.68	-2709669.43	-4381977.29	3748607.44
50464	36 26 23.060	121 55 21.299	-7.78	-2716390.44	-4360232.04	3767551.41
50465	36 26 22.644	121 55 21.259	-8.20	-2716393.43	-4360238.76	3767540.86
50466	36 26 21.924	121 55 20.672	-7.42	-2716388.32	-4360258.20	3767523.48
50467	36 26 22.114	121 55 20.954	-9.14	-2716391.71	-4360250.36	3767527.17

MAGNET Output

Table 9

read into memory from the first station tape. As the second station tape is read, a sequential adjustment is performed to the remote position after a simultaneous pass is encountered. This adjustment continues until the time period of the first seventeen passes has been scanned. At this point more data can be entered, or the computation stopped and the results output. The adjustment yields the position of the remote station relative to the control station. An approximate conversion to a local datum may be performed at this time.

A simple test of the internal consistency of the technique would be a closure test. A closure test will not detect scale or orientation induced errors. Scale and orientation differences are systematic errors caused by differences in coordinate systems and should not be considered in the evaluation of the translocation computation. Scale and orientation corrections can be made during the transformation from cartesian to geodetic coordinates or directly to the cartesian coordinates. To minimize the possible sources of error in the computations the closure test was computed in cartesian coordinates only. To help insure that an above average (or below average) data set was not used for the computations, two different data sets were used. One set was used to compute two of the three legs of the figure. Another data set, from 10 days earlier, was used to compute the last leg of the figure. The baselines computed are the sides of the triangle formed by stations 50459, 50460, and 50463 (Fig. 4). The 24 pass data set covers a time span of 5 days, and the 29 pass solutions span a 3 day period. The 5 day period is not representative of the time required to collect simultaneous pass data and was caused by non-technical problems[12]. The 3 day data set is more representative of the time required to obtain data, but is still a little lengthy.

The results shown (Table 10) are the MX-1502 derived coordinate differences. If the solution was perfect the differences would total to zero, therefore the totals shown are the error in the position determinations. If one divides the misclosure (1.35 m) into the baseline distance, the proportional accuracy (or error) is obtained. The resultant proportional error is 1:149000.

Baseline	dx	dy	dz	passes	length
50459 to 50463	-5289.55	63079.76	-66970.89	24	92152.78
50463 to 50460	-23683.17	-27709.66	47401.43	29	59796.43
50460 to 50459	28973.57	-35369.59	19570.38	29	49734.05
total	0.85	0.51	0.92		201683.26

Values are in meters

MX 1502 Traverse
Table 10

Originally, the closure test was done by using the published NAD-27 position as the initial control station position. The local datum position derived from the translocation was then used as the control station position

[12] The antenna at station 50460 was knocked over by cattle during this period. Because data was collected through the period, the questionable period (2 1/2 days) had to be rejected.

for the next translocation. This method did not produce suitable results, they were obviously in error when compared to the published geodetic coordinates. The error was introduced because the translocation computation is performed in the cartesian coordinates of the BE. To transform the local (NAD-27) position which has been input for the control station to the BE datum, a set of transformation parameters needs to be input into the receiver. The transformation parameters are generally average origin shifts and may or may not be appropriate to the area being surveyed. The origin shift values normally quoted for transformation of WGS-72 coordinates (often erroneously referred to as the BE datum) to the NAD-27 datum are dx = 22 m, dy = -157 m, and dz = -176 m. However, the data sets used to determine these means have spreads of at least 24 m, 13 m, and 16 m, respectively [Ref. 42]. The observed origin shifts for the Monterey area are dx = 28, dy = -163 m, and dz = -182 m. These values were obtained from the conversion of the PE derived coordinates, by the method shown in [Ref. 43], computation shown previously in Fig. 10. As previously stated, if positions awithin a survey area are going to be transformed between the local and Doppler datums, the datum shifts must be directly observed.

To achieve maximum relative accuracy to the local established control the best procedure is to translocate from an established geodetic control station. This allows one to difference the coordinates in the cartesian system of the BE, apply these ffferences to the published cartesian position, and then perform a transformation with no origin shift value needed. The conversion can be performed using a hand held calculator and the equations in Fig. 2. The MX-1502, and presumably all other receivers, can also be used to perform this compuation.

The baseline lengths computed for stations 50463 and 50460 relative to 50459 are shown in Table 11. These baselines were determined with the MX-1502 translocation option. The tabulation shows the increase in uncertainty as a function of passes used. The data sets represent approximately 1, 2, and 3 days of Doppler data. The drms value shown is the square root of the sum of the squares of the standard deviations. The standard deviations shown are for latitude, longitude, and height; these values are output of the translocation program. Presumably, they show the variance of the remote station while the control site is held fixed. The proportional (Prop.) error is what one would compute if the baseline vector was actually in error by the drms value shown.

Baseline	Passes	Length	drms	Prop.
50459 to 50463	15	92153.87	18.38	1:513813
	31	92153.86	14.40	1:639957
50459 to 50460	14	49733.61	25.16	1:197669
	29	49734.05	16.92	1:293936
	43	49734.51	14.72	1:337870

Baseline	Passes	s Lat	s Lon	s Hgt
50459 to 50463	15	7.0	13.0	10.9
	31	5.1	10.3	8.7
50459 to 50460	14	9.0	19.3	13.4
	29	6.5	12.0	10.0
	43	5.6	10.3	8.9

standard deviations (s Lat, s Lon, and s Hgt) and their root mean squares (drms) are in centimeters

baselines are in meters

Uncertainty vs. Passes
Table 11

X. LAKE SUPERIOR SURVEY RESULTS

A. DOPPLR RESULTS

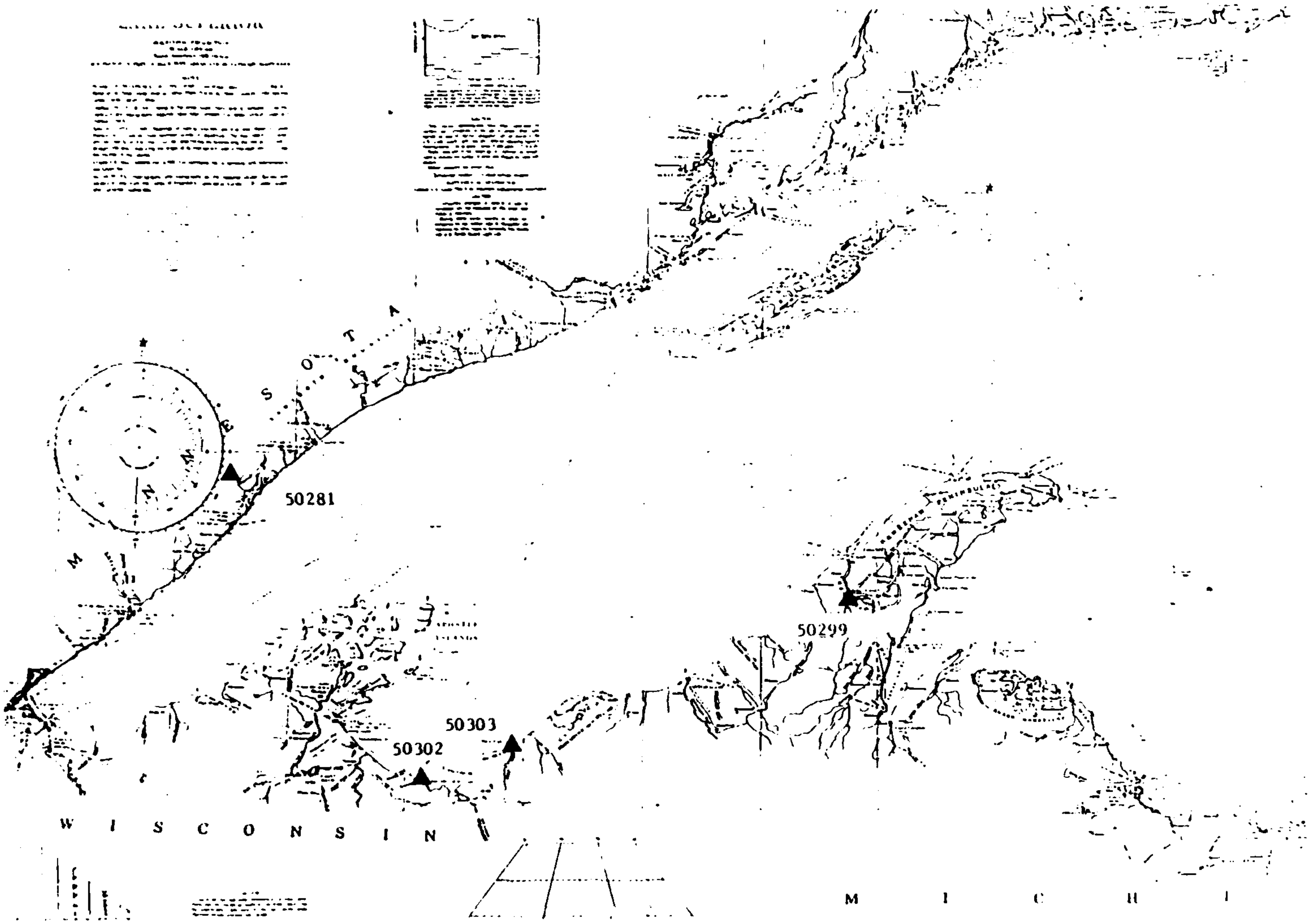
Table 12 is a tabulation of the observed datum shifts at two of the four established geodetic stations (Fig. 12) which were occupied during the Doppler survey. These two stations are at opposite ends of the survey, and on opposite shores. It is obvious from the values for the datum shift in X that the local network is not consistent. The difference between the two datum shifts is a distance of 4.88 meters, 4.97 m in the horizontal components, the resultant horizontal proportional error is 1:41600. The horizontal difference was computed from the coordinate difference values (Local-Doppler) found in Table 17. The surface shift values shown in Table 12 are the station specific values which are used to transform a point from NAD 27 to NSWC 9Z-2. Both stations are first order geodetic stations and should therefore have a relative accuracy of 1:100000. The actual accuracy would classify the relationship as second order, class II. As in the Monterey survey, much of this error can be attributed to the stations having not been established on the same survey (project). The distance between the two stations (206 km) precludes a direct tie being made via conventional methods.

Table 13 is a summary of the datum shifts observed at the other two established stations. The proportional error in this case is 1:24800, which is second order, class II relative to station 50281 (the worst case).

Because of the inconsistency in the observed datum shifts across the survey area, three sets of shifts were used to transform the Doppler point position coordinates to local datum coordinates. The application of this zoning is seen in Tables 14, 15, and 16. The zoning was performed to insure that the newly established Doppler stations would agree well with the established control in the vicinity of the Doppler stations. Doppler positions were transformed using the datum shift values observed at the base station nearest the Doppler station being transformed. This was done to minimize any errors which might occur when Doppler stations are used in conjunction with already existent control. The zoning was somewhat arbitrary in that there is no good way to determine where (or if) "jumps" in the datum occur. A diagram showing all conventional surveys made in the area would give some assistance in that one could determine which stations had been interconnected.

Table 17 summarizes the coordinate differences based on the mean datum shifts observed at stations 50302 and 50303. The spread of the coordinates (4 m in latitude and 5 m in longitude) indicates that there are considerable distortions in the local geodetic network. This situation clearly shows the need to occupy local established geodetic control while conducting a Doppler survey for hydrographic shore control. If the relationship of the Doppler control to the local control is not determined, the relationship of the hydrography to the shore will not be accurate. Occupation of established control allows the Doppler survey to be kept consistent with the local control through two methods. First, point positioning and reduction of data with the PE will yield the appropriate datum shifts which can be used to transform all Doppler coordinat Second, if relative positioning is used, the relationship to the local control can be determined independent of computing datum shifts. The computed position differences are applied to the published positions yielding Doppler derived positions consistent with the local control. An advantage to PE reduction of the known stations is that this information might be used to adjust all of the local network in a major adjustment.

Table 18 was included to demonstrate the possible spread of datum shift values. The reduction shown is of Doppler data observed in Alaska, by DMA and NGS, reduced with program DOPPLR. It was included to dramatize the need to occupy the local geodetic control when conducting a Doppler survey for establishment of hydrographic control. Obviously, use of a single set of mean datum shifts would yield significant errors if they were used to transform Doppler point positions (NSWC 9Z-2) to the local datum.



Lake Superior Existing Geodetic Stations

Figure 11

STATION NUMBER	SOL CODE	CITY OF TOWN	STATION LOCATION COUNTRY	STATE	LATITUDE(N) D M S	LONGITUDE(W) D M S	DATUM SHIFT (LOCAL- NSWC 9Z-2) DX (M)	DY (M)	DZ (M)	SURFACE SHIFT N(X) (M)	E(Y) (M)
50281	1003	FINLAND	US	MN	47 27 23	91 14 15	25.69	-147.35	-171.79	12.28	28.87
50299	1003	HOUGHTON	US	MI	47 6 44	88 33 6	30.42	-148.29	-172.56	10.16	26.66
					MEAN DATUM SHIFT		28.06	-147.82	-172.18		
					STANDARD DEVIATION OF MEAN		3.34	.66	.55		

Summary of Datum Shifts (50281 & 50299)

Table 12

STATION NUMBER	SOL CODE	CITY OF TOWN	STATION LOCATION COUNTRY	STATE	LATITUDE(N) D M S	LONGITUDE(W) D M S	DATUM SHIFT (LOCAL- NSWC 9Z-2) DX (M)	DY (M)	DZ (M)	SURFACE SHIFT N(X) (M)	E(Y) (M)
50302	1003	SAXON	US	WI	46 33 49	90 26 16	30.29	-150.26	-174.71	8.00	31.44
50303	1003	BLACK HARBOR	US	MI	46 40 3	90 2 52	31.03	-150.37	-174.24	8.09	31.15
					MEAN DATUM SHIFT		30.66	-150.32	-174.47		
					STANDARD DEVIATION OF MEAN		.52	.08	.33		

Summary of Datum Shifts (50302 & 50303)

Table 13

STATION NUMBER	SOL CODE	DOPPLER COORDINATES X (M)	Y (M)	Z (M)	TRANSFORMED DOPPLER COORDINATES X (M)	Y (M)	Z (M)	TRANSFORMED DOPPLER COORDINATES LAT (+NORTH) D M S	LONG (+EAST) D M S	ELLIP HT (M)
50283	1003	19724.92	-4282585.54	4710848.24	19750.61	-4282732.89	4710676.45	47 55 3.4686	-89 44 8.7787	185.45
50284	1003	-25111.89	-4296766.44	4697982.83	-25086.20	-4296913.79	4697811.04	47 44 42.8740	-90 20 4.1995	188.38
50285	1003	-69546.49	-4314659.29	4681221.48	-69520.80	-4314806.64	4681049.69	47 31 17.4749	-90 55 23.0814	189.82
50286	1003	-95150.31	-4333874.56	4663125.20	-95124.62	-4334021.91	4662953.41	47 16 51.3462	-91 15 26.4449	202.43
50287	1003	-126636.21	-4354867.89	4642890.98	-126610.52	-4355015.24	4642719.19	47 0 48.4783	-91 39 54.9137	188.12

REMARKS: DX= 25.69M, DY=-147.35M, DZ=-171.79M, RX= .00SEC, RY= .00SEC, RZ= .00SEC, K= .0000 PPM
COORDS REFERENCED TO NAD 1927 DATUM,
CLARKE 1866 ELLIPSOID, 8 DEGREE CUTOFF.
DATUM SHIFT AT 50281.

Transformed Doppler Coordinates (50281)

Table 14

STATION NUMBER	SOI CODE	DOPPLER COORDINATES X (M)	Y (M)	Z (M)	TRANSFORMED DOPPLER COORDINATES X (M)	Y (M)	Z (M)	TRANSFORMED DOPPLER COORDINATES LAT (+NORTH) D M S	LONG (+EAST) D M S	ELLIP HT (M)
50288	1003	-110883.13	-4374874.65	4624578.33	-110852.47	-4375024.97	4624403.86	46 46 20.4211	-91 27 5.1295	192.56
50289	1003	-105848.32	-4373456.71	4626017.63	-105817.66	-4373607.03	4625843.16	46 47 28.7315	-91 23 9.5205	185.42
50290	1003	-6973.56	-4390314.97	4611824.64	-6942.90	-4390465.29	4611650.17	46 36 6.2587	-90 5 26.1784	545.10
50291	1003	27490.14	-4371983.67	4628522.05	27520.80	-4372133.99	4628347.58	46 49 27.1905	-89 38 21.6643	186.32
50292	1003	51221.75	-4367544.12	4632484.03	51252.41	-4367694.44	4632309.56	46 52 34.7636	-89 19 39.7109	187.03
50300	1003	-84262.69	-4368377.96	4631222.64	-84232.03	-4368528.28	4631048.17	46 51 35.0490	-91 6 16.6140	186.13
50301	1003	-70149.91	-4391048.54	4610111.21	-70119.25	-4391198.86	4609936.74	46 34 57.8308	-90 54 53.3841	185.34

REMARKS: DX= 30.66M, DY=-150.32M, DZ=-174.47M, RX= .00SEC, RY= .00SEC, RZ= .00SEC, K= .0000 PPM
COORDS REFERENCED TO NAD 1927 DATUM, CLARKE 1866 ELLIPSOID, 8 DEG. CUTOFF.
MEAN DATUM SHIFT AT 50302 & 50303.

Transformed Doppler Coordinates (50302 & 50303)

Table 15

STATION NUMBER	SOL CODE	DOPPLER COORDINATES X (M)	Y (M)	Z (M)	TRANSFORMED DOPPLER COORDINATES X (M)	Y (M)	Z (M)	TRANSFORMED DOPPLER COORDINATES LAT (+NORTH) D M S	LONG (+EAST) D M S	ELLIP HT (M)
50293	1003	153757.51	-4324520.13	4670165.13	153787.93	-4324668.42	4669992.57	47 22 28.2873	-87 57 48.1832	183.40
50294	1003	89270.27	-4345883.63	4652110.75	89300.69	-4346031.92	4651938.19	47 8 6.6671	-88 49 22.3418	192.15
50295	1003	104178.92	-4337570.30	4659492.95	104209.34	-4337718.59	4659320.39	47 13 58.6545	-88 37 25.6478	184.33
50296	1003	122905.06	-4324015.18	4671537.59	122935.48	-4324163.47	4671365.03	47 23 33.9122	-88 22 17.4939	183.65
50297	1003	138789.93	-4318062.17	4676562.07	138820.35	-4318210.46	4676389.51	47 27 34.3918	-88 9 31.3526	183.45
50298	1003	161280.64	-4316015.30	4677723.20	161311.06	-4316163.59	4677550.64	47 28 29.9740	-87 51 34.7045	184.38
50304	1003	133755.22	-4340389.94	4656131.49	133785.64	-4340538.23	4655958.93	47 11 18.4715	-88 14 4.4436	182.94
50305	1003	119260.87	-4358289.67	4639885.99	119291.29	-4358437.96	4639[illegible]13.43	46 58 25.9613	-88 25 55.9015	182.88
50306	1003	117655.70	-4375901.79	4623432.89	117686.12	-4376050.08	4623260.33	46 45 26.6691	-88 27 34.2099	183.60

REMARKS: DX= 30.42M, DY=-148.29M, DZ=-172.56M, RX= .00SEC, RY= .00SEC, RZ= .00SEC, K= .0000 PPM
COORDS REFERENCED TO NAD 1927 DATUM,
CLARKE 1866 ELLIPSOID, 8 DEGREE CUTOFF.
DATUM SHIFT AT 50299.

Transformed Doppler Coordinates (50299)

Table 16

STATION NUMBER	SOL CODE	STATION LOCATION LATITUDE DIR	D	M	S	LONGITUDE DIR	D	M	S	COORDINATE DIFFERENCES (LOCAL - DOPPLER) LATITUDE (SEC)	(CM)	LONGITUDE (SEC)	(CM)	HEIGHT (CM)
50283	1003	N	47	55	3	W	89	44	9	.1303	402	.2375	493	-7
50284	1003	N	47	44	43	W	90	20	4	.1289	398	.2386	497	3
50285	1003	N	47	31	17	W	90	55	23	.1276	394	.2394	501	1
50281	1003	N	47	27	23	W	91	14	15	.1268	392	.2402	503	4
50286	1003	N	47	16	51	W	91	15	26	.1267	391	.2394	503	0
50287	1003	N	47	0	48	W	91	39	55	.1257	388	.2394	506	1
50288	1003	N	46	46	20	W	91	27	5	.0000	0	.0000	0	4
50289	1003	N	46	47	29	W	91	23	10	.0000	0	.0000	0	-2
50300	1003	N	46	51	35	W	91	6	17	.0000	0	.0000	0	-3
50404	1003	N	46	55	16	W	90	53	33	.0414	128	.1089	230	38
50401	1003	N	46	50	3	W	90	51	6	.0534	165	.0821	174	-23
50400	1003	N	46	49	[illegible]	W	90	42	43	.0294	91	.0431	91	-22
50403	1003	N	46	43	19	W	90	52	22	.0406	125	.0609	129	70
50301	1003	N	46	34	58	W	90	54	53	.0000	0	.0000	0	-4
50302	1003	N	46	33	49	W	90	26	16	-.0038	-12	.0175	37	-17
50290	1003	N	46	36	6	W	90	5	26	.0000	0	.0000	0	0
50303	1003	N	46	40	3	W	90	2	52	.0039	12	-.0172	-37	21
50291	1003	N	46	49	27	W	89	38	22	.0000	0	.0000	0	-2
50292	1003	N	46	52	35	W	89	19	40	.0000	0	.0000	0	-3
50294	1003	N	47	8	7	W	88	49	22	.0904	279	.0095	20	7
50295	1003	N	47	13	59	W	88	37	26	.0905	280	.0091	19	-1
50296	1003	N	47	23	34	W	88	22	17	.0907	280	.0085	18	-2
50297	1003	N	47	27	34	W	88	9	31	.0907	280	.0081	17	-2

(CONTINUED)

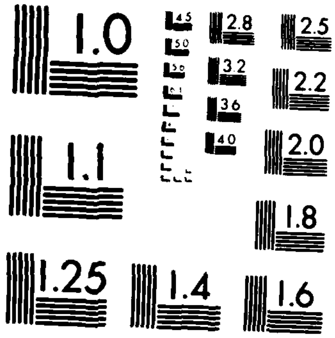
Comparison of Transformed Doppler Coordinates and Local Coordinates (Lake Superior)

Table 17

AD-A150 709 ESTABLISHMENT OF HYDROGRAPHIC SHORE CONTROL BY DOPPLER 2/2
SATELLITE TECHNIQUES(U) NAVAL POSTGRADUATE SCHOOL
MONTEREY CA D H MINKEL JUN 84

UNCLASSIFIED F/G 8/2 NL

END
FILMED
DTIC



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

STATION NUMBER	SOL CODE	STATION LOCATION LATITUDE DIR	D	M	S	LONGITUDE DIR	D	M	S	COORDINATE DIFFERENCES (LOCAL - DOPPLER) LATITUDE (SEC)	(CM)	LONGITUDE (SEC)	(CM)	HEIGHT (CM)
50298	1003	N	47	28	30	W	87	51	35	.0907	280	.0073	15	5
50293	1003	N	47	22	28	W	87	57	48	.0907	280	.0075	16	2
50304	1003	N	47	11	18	W	88	14	4	.0905	280	.0082	17	-3
50299	1003	N	47	6	44	W	88	33	6	.0903	279	.0089	19	1
50305	1003	N	46	58	26	W	88	25	56	.0904	279	.0086	18	3
50306	1003	N	46	45	27	W	88	27	34	.0902	279	.0086	18	-1
50402	1003	N	46	46	42	W	90	47	21	.0392	121	.0192	41	114
		ARITHMETIC MEAN (N = 30)								.0625	193	.0611	128	6
		STANDARD DEVIATION (RMS)								.0495	153	.0941	197	26
		SPREAD								.1341	414	.2574	542	136
		MAXIMUM								.1303	402	.2402	506	114
		MINIMUM								-.0038	-12	-.0172	-37	-23

REMARKS: DX= 30.66, DY=-150.32, DZ=-174.47, RX= .00, RY= .00, RZ= .00, K= .00PPM
DOPPLER COORDINATES: DERIVED WITH PROGRAM 'DOPPLR',
VERSION - NGS-03. 8 DEGREE CUTOFF.
MEAN DATUM SHIFT AT 50302/1003 & 50303/1003.
LOCAL COORDS REFERENCED TO NAD 1927 DATUM,
CLARKE 1866 ELLIPSOID.

Comparison of Transformed Doppler Coordinates
and Local Coordinates (Lake Superior)

Table 17 (cont.)

STATION NUMBER	SOL CODE	CITY OF TOWN	STATION LOCATION COUNTRY	STATE	LATITUDE(N) D M S	LONGITUDE(W) D M S	DATUM SHIFT (LOCAL-PRED NAD1983) DX (M)	DY (M)	DZ (M)	SURFACE SHIFT N(X) (M)	E(Y) (M)
30060	1000	SHEMYA	US	AK	52 42 55	185 52 22	38.69	-219.12	-162.29	177.89	214.01
30064	1000	ATTU I.	US	AK	52 50 10	186 48 53	39.10	216.21	-161.56	183.27	210.04
30254	1000	SHEMYA	US	AK	52 43 1	185 53 10	38.08	-218.51	-162.80	179.09	213.45
50241	1000	ST. LAWRENCE IS.	US	AK	63 41 45	170 29 3	-56.60	25.16	-202.59	53.49	-34.17
50242	1000	NOME	US	AK	64 31 9	165 22 32	10.48	-131.57	-182.79	85.88	129.96
50243	1000	TIN CITY	US	AK	65 33 35	167 56 21	10.67	-131.32	-183.11	88.55	130.65
50391	1000	ST. LAWRENCE IS.	US	AK	63 46 30	171 43 58	-57.39	27.05	-205.69	50.82	-35.02
50392	1000	ST. LAWRENCE IS.	US	AK	63 40 53	170 28 22	-56.99	25.84	-203.67	52.81	-34.91
50393	1000	ST. LAWRENCE IS.	US	AK	63 41 45	170 29 3	-57.02	26.40	-203.64	52.84	-35.46

REMARKS:

LOCAL COORDINATES: NAD 1927, ELLIPSOID: CLARKE 1866
PREDICTED 1983 COORDINATES: DOPPLER NSWC-9Z2 (NWL-9D) COORDINATES TRANSFORMED BY
RZ = 0.5 SECOND (EAST), SCALE = -0.5 PPM, DZ = 4.0 METERS

Summary of Datum Shifts (Alaska)

Table 18

XI. GEODOP V REDUCTION

All Doppler reductions performed with the GEODOP V reduction package which are included in this paper were performed on the NOAA Univac system. The Monterey Doppler data has been reduced twice; once at the NPS and then later at the NGS. The original Monterey runs were performed on the IBM 370 at the NPS. The Univac version is an adaptation of the IBM version and has been tested with test data. While at the NPS virtually all the data was reduced with a few minor exceptions. These runs were performed with all the program defaults except satellite frequency offsets. Due to the time and effort required to get the GEODOP package operational at the NPS, in depth evaluation of the results was not possible before the author was transferred to the NGS, Astronomy and Space Geodesy section.

While assigned to the NGS, two facts came to light which indicated the reductions should be performed again. First, while Mr. Archinal was conducting his thesis research at the Ohio State University, he discovered a bug in the MX-1502 data input subroutine in PREDOP. The bug caused some acceptable passes to be rejected. Since part of this paper deals with attainable accuracies versus time (number of passes), the IBM results would not be suitable for use as examples. Secondly, while attempting to get the GEODOP package operational on the NGS Univac it became obvious that some of the default values were not appropriate for MX-1502 data. The default values had been set for reduction of Canadian Marconi (CMA) receiver data since the Geodetic Survey of Canada uses mostly CMA receivers.

All of the Monterey data was not reduced again. Instead, the assumption was made that the default values had biased all of the results equally and therefore, only the data sets which had been selected after the initial runs at the NPS were reduced on the Univac. The reductions were performed at the NGS after the appropriate software changes had been made and tested. After discussions with Mr. Kouba, appropriate default values and reduction procedures were implemented and the pre-selected subsets of the Monterey data were reduced.

Because this reduction program is so complex it can be difficult to decide which reductions are representative. In an attempt to maintain consistency, a standard procedure was developed for data reduction. This procedure was used in the reduction of all data, and it is described in Appendix C. One of the single most important indicators of the validity of a solution is the estimated standard deviation of unit weight (SO) for each station. Additionally, the spread of all the SO's should be less than 0.10. Unless otherwise indicated no solution was presented in this paper with station SO's less than 0.90 and a spread greater than 0.10. Most solutions had values between 0.90 and 0.95. The optimum value is 0.95 for the individual SO's. Solutions with these values should have the most accurate baselines and estimated standard deviations of the position differences. These sigmas are used to compare the relative accuracy of solutions and therefore need to be accurate.

In the reduction procedure used, some program options are left at the default values while others such as receiver delay, satellite frequency offset, and range rate sigmas are specified. In addition, the procedures specified in the GEODOP USER'S GUIDE are also observed. To reduce the number of iterations required, the estimate of each station position was the DOPPLR derived station position. This allowed enforcement of rigorous rejection criteria on the first reductions. Normally, the first reduction is performed with relaxed rejection values to refine the estimated station positions.

The data sets from the Lake Superior survey which are used as examples in the following sections were selected before the relative position reductions had been performed. The selections were made so that the reductions could be compared to similar reductions made on the Monterey data. The differences observed would primarily be due to the difference in survey size. In all cases the results of the reductions met expectations which had been formed based on personal experience and conversations with others.

Unless otherwise noted, the assumption has been made that all pass data is of acceptable quality. Though it is unlikely that "bad" data would go undetected the possibility does exist. A systematic difference affecting the BE of all satellites could cause differences between like data sets seperated in time. Because there were no significant differences observed between data sets it is doubtful that any poor data was collected during either survey. The only way to guarantee that there were no BE induced errors in the solution variances would be to perform the GEODOP V reduction with the PE.

A. EFFECT OF NETWORK CONFIGURATION

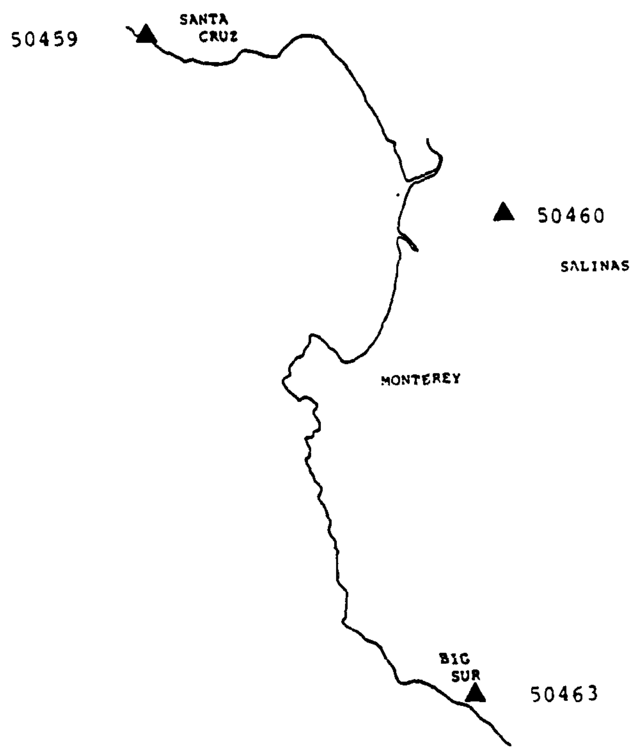
To observe the effect of network configurations (survey geometry) on the internal relative uncertainty of a survey the best and worst cases of both the Monterey and Lake Superior surveys were reduced. Based on conversations with Mr. Kouba, network configuration should not have a major effect on either survey since the baselines are less than 500 km. This value was based on experience with Doppler data collected and processed by the Geodetic Survey of Canada. As was expected, a small effect was observable in the Lake Superior reduction.

1. Monterey survey

The two station configurations compared are the triangle formed by stations 50459, 50460, and 50463 (Fig. 12) and the linear configurations of stations 50459, 50461, and 50463 (Fig. 13). Both solutions have approximately the same number of pases with 29 and 31 passes, respectively. The sigmas of the positional differences are used as the indicator of relative positional uncertainty.

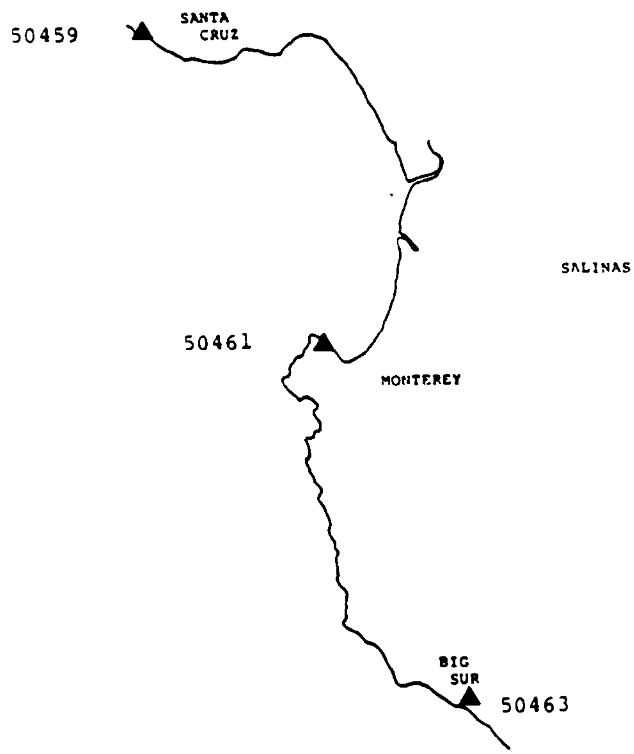
The data sets shown, demonstrate the importance of network configuration on surveys with small baselines. The differences in the sigmas of the position differences is about 1 cm in each axis (Table 19). This difference is not significant and cannot be attributed to network geometry. The difference could easily be attributed to the two examples being based on different data sets.

The baseline distance between station 50459 and 50463 does not agree well between solutions. The DOPPLR derived baseline of 92,153.07 m (Table 8) is within 12 cm of the mean of the GEODOP results (92152.95) shown in Table 19. The difference in baseline length is due to the second reduction being performed with the wrong receiver delay. The difference could be reduced by performing another reduction with the correct delay value used in the reduction. However, the proportional error (assuming the difference is the error) is still acceptable at 1:256000.



Triangular Station Configuration (Monterey)

Figure 12



Linear Station Configuration (Monterey)

Figure 13

FROM	TO	dx	dy	dz	baseline
50459	50463	22.96	23.64	15.71	92153.13
50459	50463	20.85	15.64	14.54	92152.77

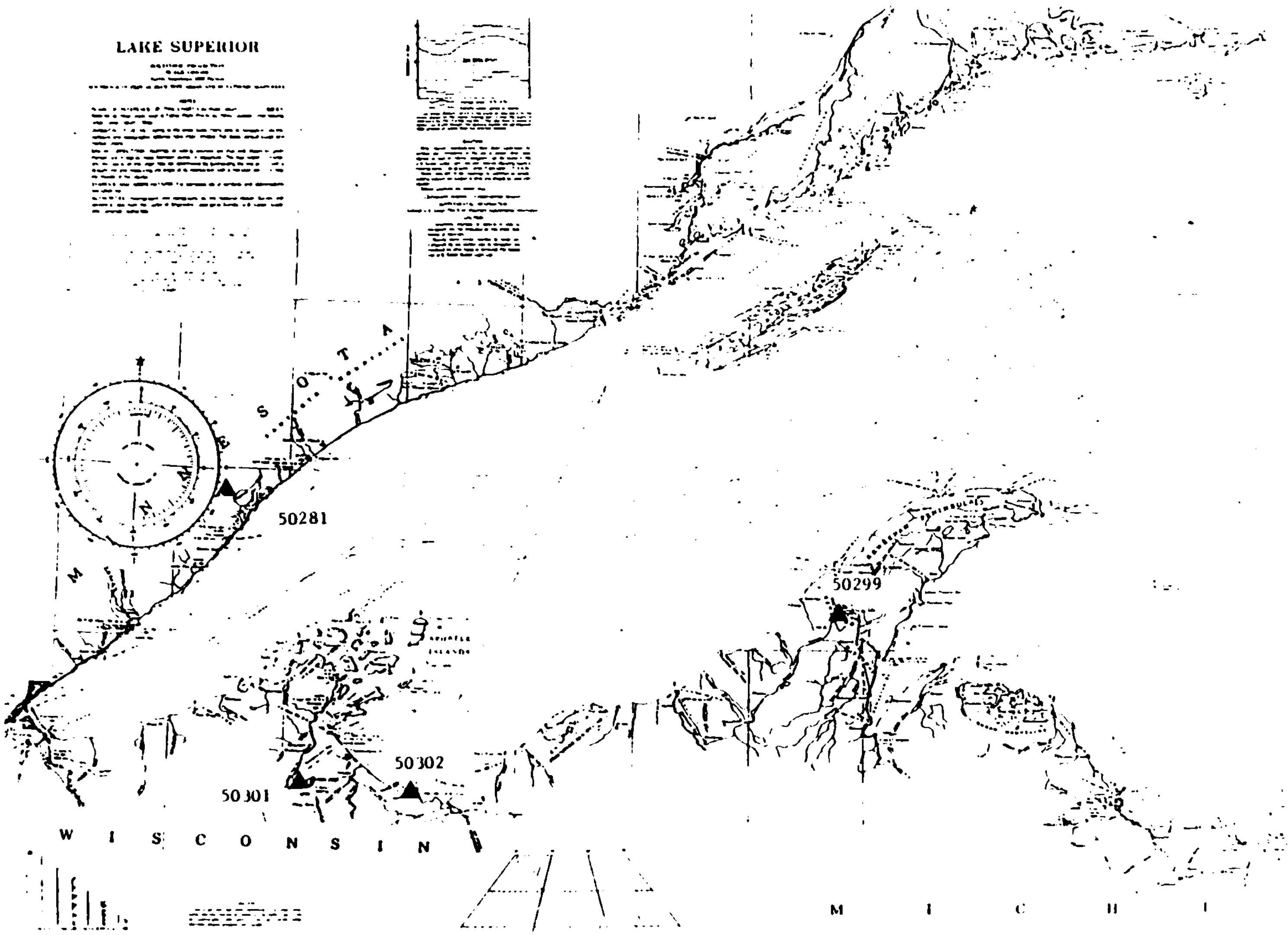
First case (50459-50460-50463) shown in Fig. 13
Second case (50459-50461-50463) shown in Fig. 14
Uncertainties (dx, dy, & dz) in cenitmeters, baselines in meters

Monterey Network Configuration
Table 19

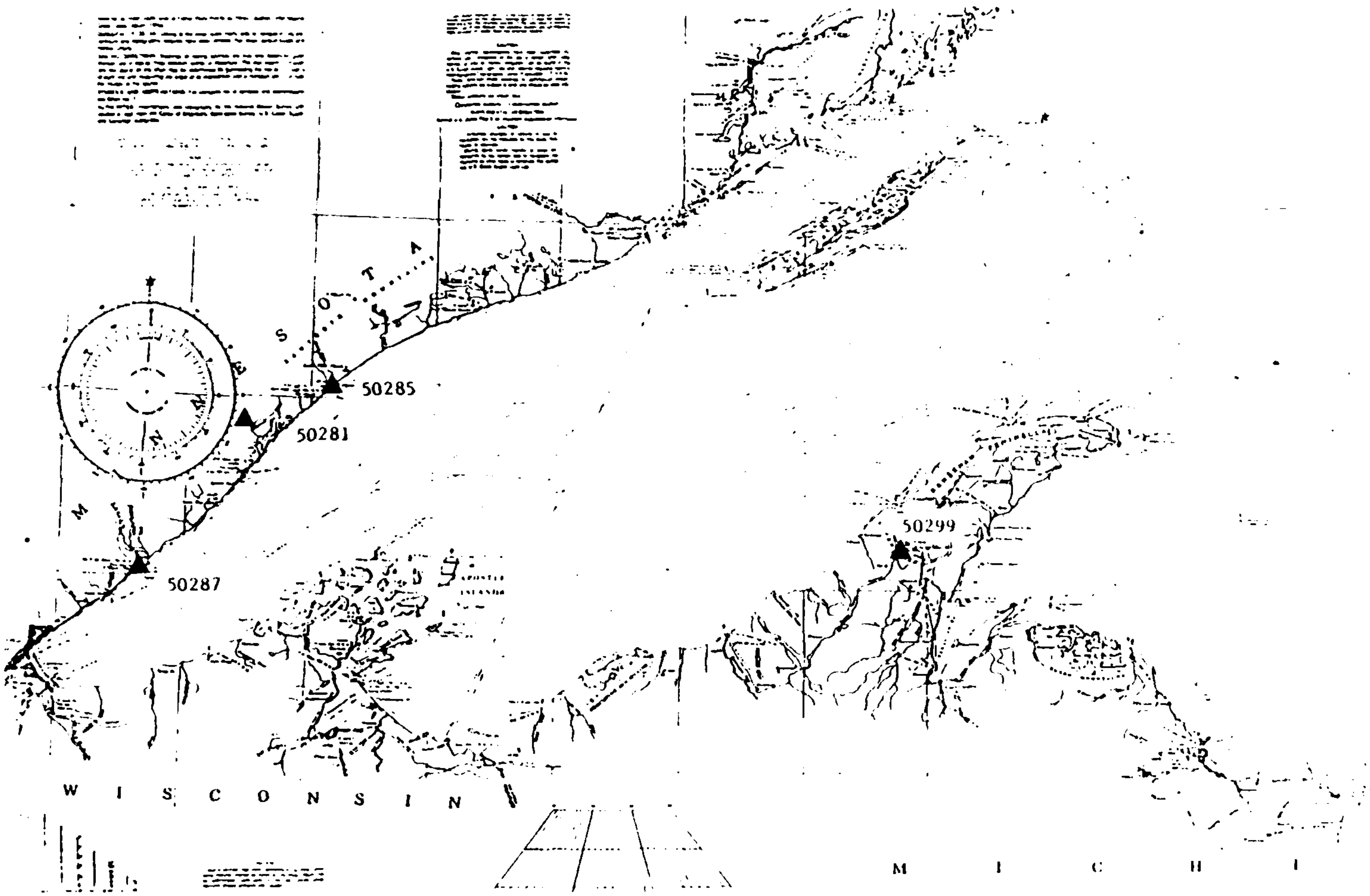
2. Lake Superior survey

Unfortunately, the data sets available for comparison are not as extreme as would be desired for demonstrative purposes. Because this survey was operational in nature, and not for research, stations were occupied based primarily on logistical concerns. The best possible configuration would have been a quadrilateral surrounding the lake, stations 50281, 50283, 50291, and 50299 would have formed this figure. This combination is based on using the same base stations as were used for the survey. The example used for the worst case is the worst case that could have been constructed from the occupied stations. Figures 14 and 15 show the best and worst case examples, respectively.

Referring to Table 20 it is apparent from the deviations of the position differences that the best solution for station 50299 is obtained from the first data set. Both data sets have approximately the same number of passes; the difference not being large enough to explain the differences in the sigmas of the position differences. Both data sets reflect approximately two days of station occupation. Note that the baseline distance (50281 to 50299) did not change significantly (.44 m, 1:470000) between the two solutions.



"Best" Survey Configuration
Figure 14



"Worst" Survey Configuration
Figure 15

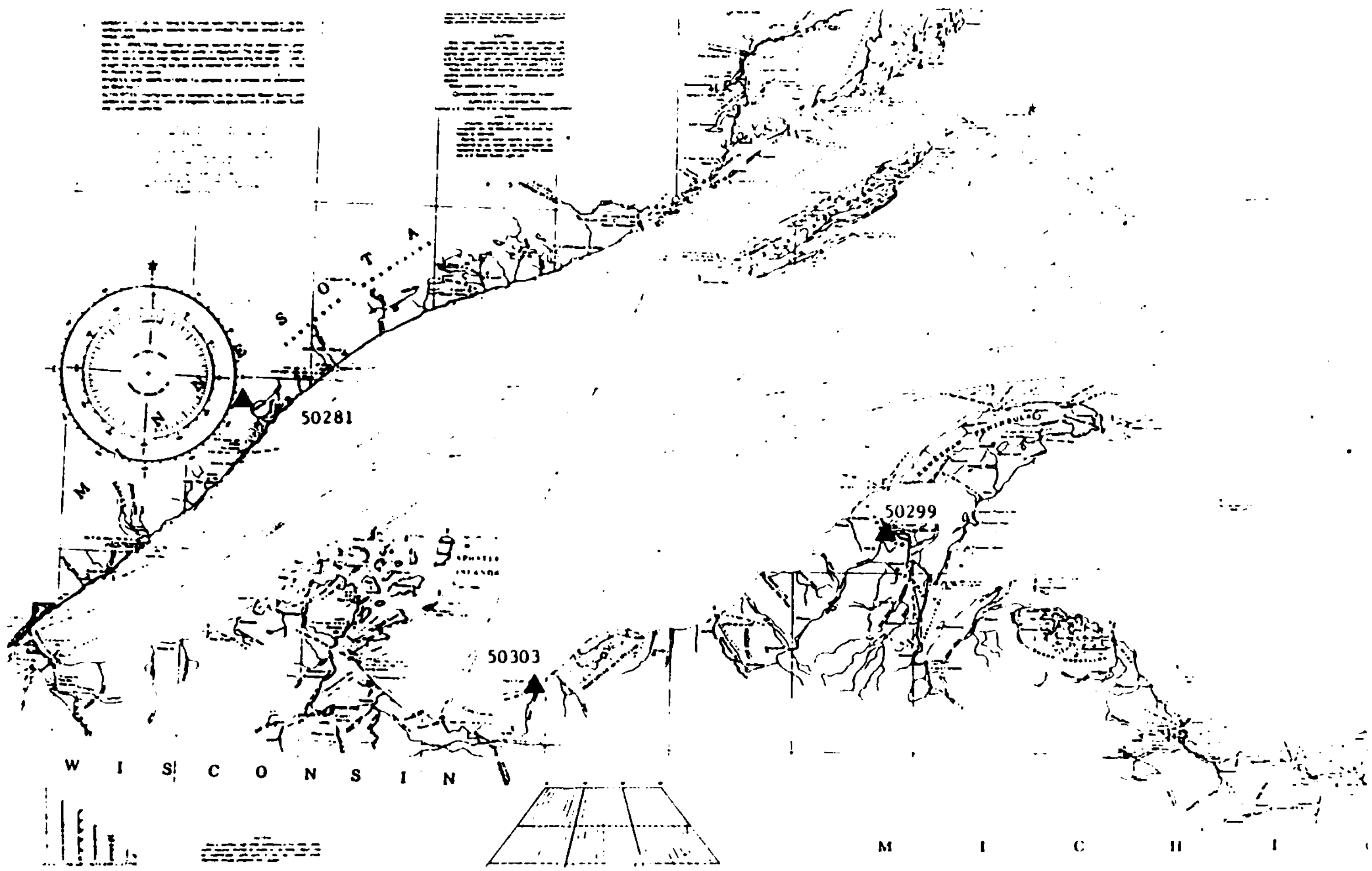
FROM	TO	dx	dy	dz	baseline
50281	50299	22.75	15.38	15.14	206770.41
50281	50301	18.10	11.88	11.30	
50281	50302	21.38	13.92	12.89	
50281	50299	28.70	18.91	18.69	206769.97
50281	50285	23.10	12.69	11.88	
50281	50287	26.55	13.29	13.03	
50281	50299	24.32	16.45	16.18	206770.17
50281	50303	21.97	13.44	13.23	

baselines are in meters
uncertainties (dx, dy, and dz) are in centimeters

Lake Superior Network Configuration
Table 20

The additional solution included in Table 20 (last case) was included as an example of what may be gained by a single additional station (Fig. 16). The solution variances are near the variances in the best case (2 cm in each axis). The difference is due to the first case having more data than the last case. In the third case all but one of the 34 passes were common to all stations; in the first and second cases this high commonality is not seen. This is the reason for the last case having similar uncertainties as the first case. The solutions of the first and second cases would be improved if a higher pass commonality existed. The first case had the lowest pass commonality. Therefore, it's solution would improve the most.

The results shown here indicate that network configuration may have an effect on the solution uncertainties of large surveys. Surveys with



Three Station Configuration
Figure 16

baselines of 500 or more kilometers will be affected by network configuration[13]. In either case, large or small survey, network configuration must be considered to obtain the best tie to the local geodetic control.

B. ADDITION OF MORE DATA

The accuracy of a Doppler position is inversely proportional to the number of passes observed. The solution usually reaches convergence at 30 to 40 passes for point positioning reductions. In most parts of the contiguous 48 states, 40 passes can be observed in 3 days or less.

Comparing the results from the GEODOP V reductions shown in Table 20 with the results of the equation quoted from [Ref. 44], SIGMA = $150/(N(S-1))^{1/2}$ one sees little difference (Fig. 18). The GEODOP V results shown are not optimum but are acceptable for this comparison. Therefore, one could use this equation to determine how many passes are needed to obtain a specified uncertainty. There are two factors which do need to be considered if this equation is to be used for predictive purposes; 1) only common passes are used in the equation and 2) the approximation assumes good data. In the field, one can either add additional passes as a safety margin or review the data at all sites to verify the number of acceptable common passes. The number of 3-D passes (MX-1502) could be used as an indication of the number of acceptable passes. The accuracy of the prediction may go down as one deals with smaller data sets (10 or less passes); especially as the number of observing stations decreases.

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[13] Personal conversation with J. Kouba, Geodetic Survey of Canada, 1983.

FROM	TO	N	dx	dy	dz
50459	50460	40	17.13	15.10	11.59
50459	50463		17.33	15.33	11.81

$$\text{SIGMA} = 150/(N(S-1))^{1/2}$$

N = 40 passes

S = 3 stations

SIGMA = 16.77 cm

FROM	TO	N	dx	dy	dz
50222	50231	14	28.91	23.31	18.93
50222	30691		27.65	23.91	19.10

SIGMA = 28.35 cm

Uncertainties (dx, dy, & dz) are in centimeters

Comparison to Estimate
Figure 17

To estimate the obtainable precision of point position, precise ephemeris reductions with respect to the number of passes observed one can use the equation in Fig. 18. This equation is used in program CLASSI to determine the a-priori accuracy estimates of Doppler positions weighted to the number of passes [Ref. 45]. The weighting is based on results from 30 pass data sets. This equation also assumes high data quality; passes with range rate sigmas greater than 0.30 m cannot be used. The associated sigma values are based on experience at NGS in performing adjustments. There are two groups of values for the sigmas presented. The first group is for use when observations are going to be performed in a relatively short time period (less than 1 year). The second set is used when observations are going to be performed over a longer period. The higher value is required due to the long term variation in the PE coordinates as reported in [Ref. 46].

During the GEODOP V reduction of the Lake Superior survey, five 15 station reductions were performed. These reductions were primarily performed to test the program since it had been recently adapted to the NOAA Univac. The results are included only to show that there is a point where added passes do not significantly improve the solution. Comparing the sigmas of position differences between stations 50281 and 50299 (Fig. 19) to the sigmas of the first solution shown in the FGCC data set (Table 21) one observes little difference. The solutions are based on 255 and 78 passes in common, respectively. In fact, little difference is seen between the first and second cases of the FGCC results. Solution convergence for a point position solution is obtained at 40-50 passes. These results indicate the same is probably true for relative positioning, obviously it is reached at 70 passes.

Due to periodic, systematic variation of the BE it is best to reduce data sets of 100 passes or less[14]. Larger data sets could be affected by this variation. The best technique for reduction of large surveys is to reduce the data sets in small groups, then perform an adjustment of the entire survey using the subset solutions. The most practical method for subdivision of the survey data set is to reduce each group of stations which were observed simultaneously. The size of these data sets will be determined by the number of receivers used in the survey. One must bear in mind that to perform the adjustment all stations must be linked to one another through

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[14] Personal conversation with J. Kouba, Geodetic Survey of Canada, 1983.

Based on a-priori estimates of 30 pass solutions, for long term data sets, (1 year or more), where:

σ_{30} = 60, 80, and 100 cm (ϕ, λ, & ht.)

passes	σ_ϕ	σ_λ	σ_h
10	103	139	173
15	85	113	141
20	73	98	123
25	66	88	110

$$\sigma_w = \frac{\sigma_{30}}{(\text{Total passes}/30)^{\frac{1}{2}}}$$

$$\sigma_w = \sigma_\phi,\ \sigma_\lambda,\ \&\ \sigma_h$$

σ_{30} is the standard deviation in a coordinate axis based on 30 passes, σ_w is the resultant estimated standard deviation in thecoordinate axes (σ_ϕ, σ_λ, & σ_h).

For short term data sets, σ_{30} = 30, 40, & 40 cm.

passes	σ_ϕ	σ_λ	σ_h
10	50	70	86
15	42	66	70
20	37	49	49
25	33	44	44

Note: All values are based on a reduction with an 8° cutoff, and at least on pass in each quadrant.

Point Position Accuracy Estimate

Figure 18

From	To	passes	dx	dy	dz
50281	50299	255	11.46	7.62	7.46
50222	50231	78	12.48	8.17	7.43

Uncertainties (dx, dy, & dz) are in centimeters

Sigma Differences
Figure 19

one or more stations. These linking stations can and should be the base stations. Additionally, all mobile stations should be moved at the same time. If on a four receiver survey two remotes were moved on alternate days, one could not subdivide the data sets into 4 station groups without cutting a station's pass data in half.

XII. STATION ELEVATION DETERMINATION

The compuation of station elevation from Doppler measurments is dependent on an accurate knowledge of the geoid. The geoidal slope of the area should also be known for the survey area if the results are to be optimum. This is the reason for the emphasis on occuping base stations which have ties to the NGVD. Because Doppler positions yield an ellipsoid height at each station one can determine the geoidal height to a reasonable approximation. The accuracy of the approximation is affected by the accuracy of the tie to the NGVD. By subtracting the elevation at a station from the ellipsoidal height one computes the geoidal height. Comparison of all geoidal height values at base stations allows one to infer the geoidal slope of the survey area. This inference can be degraded by large changes in the topography of an area.

Assuming one can use the geoidal slope information obtained for the survey area, each station in the area can be corrected to yield an estimated elevation above MSL. These elevations can then be differenced to yield height differences between stations. This method (Fig. 20) is not accurate enough to replace geodetic leveling but should suffice for correcting the slope ranges of the hydrographic positioning system to horizontal ranges.

As suggested earlier (sect IV. B.) if base stations with ties to the NGVD can not be found, occupation of bench marks will yield this geoidal slope information. Bench marks should be selected to bracket this Doppler survey so that the inference of the geoidal slope is best suited for the survey.

Ellipsoidal height - Elevation = Geoidal height

STATION	E.h.	Elev.	G.h.
50459	-1.87	31.93	-33.80
50464	-12.95	21.25	-34.20
50461	-27.68		

dGh = -34.20 - (-33.80)
dGh = -0.40 m

Baseline distances:

50459 to 50464 = 63383.45
50459 to 50461 = 45552.98

distance from 50459 to 50460 is approximately 72% of distance to 50464; then, assuming a constant change in the geoidal slope:

dGh = (-.40 m) (.72) = -.29 m
Gh (50461 = Gh (50459) + dGh
Gh (50461) = -33.80 + (-.3) = -34.1 m

Elevation (50459) = Ellipsoidal height - Geoidal height
Elevation (50459) = -27.68 - (-34.1) = 6.4 m

Note: This method was used since there were two stations with known elevations. If there is only one station with an elevation, one must either assume a constant (level) geoid for the survey area or use some other means to determine the geoidal slope in the survey area.

Station Elevation Computation
Figure 20

A geoidal contour map can be used to indicate areas where there might be major variation in the geoid. However, most geoidal height maps do not have sufficient resolution to allow their use for obtaining geoidal height information.

Another method of obtaining geoidal height information is with NGS' program MCANAL. The program accepts latitude, longitude, and elevations for points of interest and outputs geoidal height information for these locatons[15].

[15] Personal conversation with M. Chin, Gravity, Astronomy, and Space Geodesy Branch, NGS.

XIII. FGCC TEST NETWORK RESULTS

During the FGCC test of the Motorola Mini-Ranger Doppler Satellite Survey System in May, 1982 observations were also performed with three MX-1502's. The data that was collected by the MX-1502's was reduced with GEODOP V and is presented here as an example of attainable precision. The MX-1502 data was used for this reduction only because there is no input subroutine for Motorola data.

The three station data set was processed using the procedures outlined in Appendix C. Simultaneity of pass data was enforced in all solutions presented. This did not cause much loss of data since all stations had good horizon visibility so nearly all passes were tracked at all three stations. Numerous runs were performed to optimize the results, selection of the representative solutions was based on the formal statistics of the solutions. Meterological data was not input.

The baseline distances between the three solutions are known to a high accuracy. A conservative estimate of the estimated accuracy between the stations is 1:500000 (2 sigma) [Ref. 47]. All three stations have been tied (with first order methods) to the Transcontinental Traverse (TCT) network which has an estimated accuracy of 1:1000000. The conservative figure of 1:500000 yields an uncertainty of about 8 cm for the 42 km baseline, 7 cm for the 35 km baseline, and 4 for the 19 km baseline.

Table 21 shows the number of passes in the solution, the sigma of the position differences, and the differences between the terrestrial standard and the GEODOP V derived baselines. Also included is the estimated standard deviation of unit weight (SO) for each station. The optimum solution would be with each station having an SO of 0.95.

Some of the results should not be taken to be representative of attainable accuracy based on the number of passes. Specifically, the 5 pass solution; time and resources did not allow more research into representative accuracies for such a small data set. The solution shown was the only acceptable solution, based on the formal statistics, out of approximately 15 runs. The magnitude of the baseline differences for this set should not be considered typical for such a small data set.

The magnitude of the baseline differences of all solutions are reflected in: 1) the proximity of the station SO's to .95 and 2) the spread between the SO's. Based on the SO's and the sigmas of the position differences the third, fourth, and sixth solutions would need to be redetermined before the results would be acceptable. The sixth solution appears, based on the SO's, to be an acceptable solution. Comparison with the standard shows otherwise. The variance for the solution is high based on the number of passes. This tends to indicate a weak solution and would be sufficient cause to rerun the reduction. Bear in mind that the worst error in this data set yields a proportional accuracy of 1:95000 (first order is 1:100000). The worst case presented in Table 21 shows a proportional error of 1:55000, this would be acceptable as second order, class I. Again, these are the worst cases and would have been reprocessed, based on the statistics, if time and resources would have permitted.

Passes	Position Differences (cm) σx	σy	σz	Baseline Differences (m) 42 km	35 km	19 km	Period of Observations	SO's
78	12.48 12.48	8.17 8.65	7.43 7.81	-0.124	-0.018	-0.081	137/1900 to 142/2000	.93, .92, .92
40	17.21 16.35	12.37 12.83	10.75 11.10	0.146	0.022	0.069	140/1000 to 142/2000	.94, .92, .91
39	19.92 19.92	11.98 12.45	11.55 11.73	-0.238	-0.424	-0.111	137/1900 to 140/1000	.91, .87, .88
21	27.29 26.48	14.50 15.03	15.53 15.30	0.472	0.636	-0.021	138/2000 to 139/2400	.89, .89, .89
24	20.41 19.26	16.26 16.52	14.37 14.36	0.122	0.056	0.129	140/0200 to 141/1000	.93, .90, .90
19	25.20 27.08	17.28 18.23	14.97 15.93	-0.398	-0.364	-0.021	141/0400 to 142/1000	.92, .93, .90
14	28.91 27.65	23.31 23.91	18.93 19.10	-0.088	0.234	0.179	140/2100 to 141/2100	.93, .90, .94
5	66.71 66.63	32.38 34.44	32.85 34.39	-0.048	0.016	-0.071	142/1000 to 142/1800	.83, .88, 1.03

FGCC Results
Table 21

Of the three baselines computed, the third had the least change from one solution to the next and generally showed the best agreement to the standard. The proportional error varied from 1:104000 to 1:884000.

The data is presented to indicate the capability of Doppler methods to establish control of sufficient accuracy for use as hydrographhc shore control. Because of the length of the baselines the proportional error was low. The reader is reminded that the error associated with Doppler observations is relatively free of a proportional component. If specifications for a survey are written in terms of proportional error, the surveyor must be more concerned with the short lines, rather than the long ones.

XIV. COMPARISON OF REDUCTION METHODS

Table 22 is a summary of the baseline lengths determined by the various reduction methods. GEODOP V reductions are based on two day (approximately 30 passes) data sets. The MX-1502 results are also based on two day data sets. The MAGNET and DOPPLR results are from reduction of all available data (as much as 465 passes at a station).

Sta	MAGNET	GEODOP V	DOPPLR	MX-1502	LOCAL
50460	49733.63	49733.67	49733.65	49734.05	49732.83
50461	45551.47	45551.40	45551.38		45551.00
50462	81723.73	81723.78*	81723.73		81723.84
50463	92152.99	92153.13	92153.07	92152.78	92152.98
50464	63381.41	63381.52	63381.41		63381.66

Lines are in meters from station 50459
*Station 50462 baseline computed via station 50461

Baseline Comparison
(From station 50459)
Table 22

It is readily apparent that the Doppler results are very consistent when reduction methods are compared with one another. This consistency points out the advantages of relative positioning and the waste of adding passes past the convergence point of 30-40 passes.

The MX-1502 derived baselines for stations 50463 and 50460 show poor agreement with the other Doppler results due to poor data. The exact cause of the disagreement is not known. GEODOP V reductions performed on the same data set indicate that there was some form of receiver induced problem at station 50459. The GEODOP V reduction showed high variance of the receiver delay. The GEODOP V results presented here are from reduction of another data set. The proportional error, using the DOPPLR determined baseline as the standard, is 1:318000. The questionable data was not obvious with the MX-1502 since it does not output the receiver delay.

Table 23 shows the mean and standard deviation of the baseline lengths with and without the local control included. The difference shown (dbl) is the mean of the Doppler baselines minus the terrestrial baseline. Comparing the Doppler derived baselines to the local (terrestrial) baselines one notices a change in sign of the differences. This indicates a lack of consistency in the local geodetic network since most of the baselines are in the same direction. Because the baselines are in the same direction a difference in orientation of coordinate systems will not explain the change in sign. Most likely, these differences are the result of the local stations not having direct ties to one another.

Station	Mean	Sigma	dbl	Mean	Sigma
50460	49733.75	0.20	+0.92	49733.57	0.45
50461	45551.43	0.04	+0.43	45551.32	0.22
50462	81723.72	0.03	-0.12	81723.77	0.05
50463	92152.99	0.15	+0.04	92152.95	0.16
50464	63381.44	0.07	-0.22	63381.50	0.12

The first mean is only Doppler baselines, the second mean is both Doppler and Terrestrial, the sigmas are the standard deviations for each of the means.

Baselines are from station 50459 in meters.

Doppler-Local Comparison
Table 23

XV. LAKE SUPERIOR SURVEY EXPENSES

The Lake Superior Survey is a splendid example of how cost efficient a Doppler survey can be when compared to a conventional survey. The survey is a case of extremes in that it was very well suited for Doppler and very poorly suited for terrestrial methods.

Accessibility was poor but did not require the use of helicopters. Many times areas selected for Doppler surveys are so remote that helicopters must be used. Because conventional transportation could be used, a major expense in many Doppler surveys was not incurred on this survey. The terrain of the area was what made the area so unsuited for conventional methods. Not only was intrastation visibility poor or non-existent, the terrain was relatively flat. The lack of hills would have forced the construction of observation towers for virtually every station. The requirement for towers would have made a conventional survey extremely expensive in both the time and man-power required.

Table 24 shows a breakdown of the various expenses for the survey. Transportation is not included as the information was not available. It is a valid assumption that transportation costs would be the same for either a Doppler or conventional survey in this area. Salary costs were based on the salaries of the four men on the field party. The salary expenses are slightly high since they are based on actual expenses. Presumably a permanent field unit would be partly composed of personnel of lower grades. Per diem and overtime expenses were based on costs incurred by the two civilians on the field party, the values are for four men. The miscellaneous category covers supplies, etc. which were required for station establishment and occupation.

Salaries (Avg'd)	6800.00	
Per Diem	7524.00	(Based on actual for 2 men)
Overtime	7074.00	(Based on actual for 2 men)
Data Tapes	270.00	
Lease Fees (3 units)	21000.00	
Misc.	566.00	
Total	$43,234.00	

Lake Superior Expenses
Table 24

Based on the total shown, the per station cost of this survey is $1730. The reader is reminded that this is for the field work and does not represent the total cost since data processing costs are not included. Even still, the cost is considerably lower than operating a field unit of 8 to 12 men for a year.

XVI. SUMMARY

As is the case with all surveying systems, each is most advantageous for use in certain circumstances. Doppler is no exception to this rule. Survey areas which have plentiful, easily accessed established geodetic stations are not best suited for Doppler methods. Conventional methods would normally be better suited. Surveys conducted for the establishment of high density, high order control may also be better performed using conventional means. Doppler methods excel where established control is sparse, intrastation visibility poor, and station spacing is at least a few kilometers. Doppler methods are especially well suited for making high precision, long distance ties between local geodetic networks. Especially when distance (or topography) precludes a conventional tie being made.

It has been shown that relative positioning techniques will usually be most suited for establishment of hydrographic shore control. The improved relative accuracy and ability to do high accuracy position determinations in the field make it superior to point positioning techniques.

The proposed specification has been written to meet the present standards for hydrographic shore control for both the NOS and the IHO. The specification yields third order accuracy in the proportional sense while having an acceptable positional uncertainty. The positional error of the shore station was shown to be insignificant in respect to the errors contributed by the ranging systems usually used for hydrographic control.

Simply stated, the specification is:

1) Use FGCC field procedures to yield a 70 cm positional precision if the data are to be reduced in a point position reduction with the PE, and on all base stations regardless of reduction method.

2) If a relative position reduction is to be performed, use field procedures specified for a 50 cm. uncertainty to occupy all new stations.

3) Adjust station spacing to meet third order, class I specifications based on the reduction method to be used.

The need for the capability to compute the total error in a hydrographic position will become more pressing as navigation is done with high precision satellite systems such as GPS. The proposed differential GPS system [Ref. 48] shows great promise for both hydrographer and mariner. It will pose a problem for the hydrographer in that the mariner will be using the same system for navigation that the hydrographer uses for positioning. This will require that the stated positional error be an accurate representation of the total possible error in the hydrographic position.

Many may feel that to invest in Doppler equipment at this late data would be wasteful with GPS "just around the corner." The first geodetic GPS receivers were to be delivered to NGS and DMA in April 1983, they still have not been delivered (December, 83), and the delivery date is still conjecture. A single point positioning GPS receiver presently costs approximately $130 k. The major advantage to GPS is that the required occupation time is considerably reduced (1-3 hrs) while obtaining the same (or better) accuracy. This is not as advantageous as it seems at first.

To establish a shore control station requires that reconnaissance be performed to select a suitable station site. Once a site is selected, the property owner's permission must be obtained, the station mark set, and a description written. Based on the field experience of the author and others[16], the selection and monumentation of a suitable site takes approximately two days. This estimate is based on areas where Doppler would probably be used; i.e., remote areas with poor accessibility. In areas such as this, the survey party can be performing the reconnaissance of future stations while the Doppler unit(s) are locating other stations. Because of the required groundwork to find a station, a shortened observation period is not a significant advantage.

Another consideration in regard to GPS versus Doppler is that the TRANSIT system is up and operational. The GPS system only has six operating satellites currently. Availability is less than 12 hours per day and times of observation may vary as the satellite orbits precess. Additionally, service will not be reliable due to testing of the spacecraft and alteration of orbits during the initial period of the system.

The proper application of a GPS system in hydrography is not as a means of establishing control, but as a means of being free of the constraints imposed by shore control. The proposed GPS differential system shows great promise in this regard.

Development of interferometric reduction programs could cut Doppler observation time to approximately 6 or fewer hours, while still meeting the proposed specification. The reduced observation period, combined with the lower priced, more available Doppler equipment would make Doppler methods very competitive with GPS systems.

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[16] Personal conversation with G. Frederick, Operations Div., AMC.

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APPENDIX A. PROPOSED FGCC SPECIFICATION REVISIONS

Revised: Nov 1 1983

SATELLITE DOPPLER POSITIONING

Satellite Doppler positioning is a three dimensional measurement system comprised of observations of the radio signals of the U. S. Navy Navigational Satellite System (NNSS), commonly refered to as the TRANSIT system.

The Doppler observations are processed to determine station position in Cartesian coordinates (X,Y,Z) and can be transformed to geodetic coordinates (geodetic latitude and longitude, and height above reference ellipsoid). There are two methods by which the station position(s) can be derived; these methods are point positioning and relative positioning.

Point positioning, for geodetic applications, requires that the processing of the Doppler data be performed with the precise ephemerides. The ephemerides describe the satellites' positions in space. The precise ephemerides are computed from Earth based tracking data and are supplied by the Defense Mapping Agency. In this method, data from a single station is processed to yield the station coordinates.

Relative positioning is possible when two or more receivers are operated simultaneously in the survey area. The processing of the Doppler data can be performed by four modes: simultaneous point positioning, translocation, semi-short arc, and short arc. Only simultaneous point positioning requires use of the precise ephemerides for geodetic surveys. The other methods may or may not use the precise ephemerides. In the modes of simultaneous point positioning and translocation, the orbital coordinates are held fixed in the processing. Semi-short arc allows up to 5 degrees of freedom in the ephemerides; short arc allows 6 or more degrees of freedom.

The precisions quoted in the following sections are based on the experience gained from the analysis of Doppler surveys performed by agencies of the federal government. Since the data is primarily from surveys performed within the continental United States (CONUS), the precisions and related specifications may not be appropriate for other areas of the world.

Network Geometry

The order of a Doppler survey is determined by: the spacing between primary stations, the order of the base stations from which the primaries are established, and the method of data reduction which is used. The order

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and class of a survey can not exceed the lowest order (and class) of the base stations used to establish the survey.

The primary stations used to define the order of the Doppler survey will be selected by the surveyor. These stations will be spaced at fairly regular intervals, which will meet or exceed the spacing required for the desired accuracy of the survey. The primary stations will carry the same order as the survey.

Supplemental stations may be established at the same time (same survey) as the primary stations. The order (and class) of these stations will be determined by the spacing between the supplemental station and the nearest doppler station or other horizontal control station. Both the distance to, and the order of the nearest station will determine the order of the station, with the lowest order being assigned to the supplemental station. The method of data reduction will determine the allowable station spacing.

Station Spacing

The station spacing of Doppler stations determines the order of the survey. The minimum distance, D, may be computed by a formula defined by the type of data processing to be used. This distance is also used in conjunction with established control, and other Doppler control, to determine the order and class of the supplemental stations.

By using the appropriate formula, one may construct tables showing station spacing as a function of point or relative position precision (σ_r or σ_p) and desired survey (or station) order. The estimates for the precision are based on long term repeatability studies and comparison with standards of equal or greater precision.

Base Stations

Whenever new stations are to be established in a given survey, one must occupy, using the same Doppler equipment and procedures, at least two existing horizontal network (base) stations having datum values certified as having an order (and class) equivalent to, or better than the intended order of the Doppler survey. If the Doppler survey is to be first order, at least three base stations must be occupied. If relative positioning is to be used, all base station baselines must be directly observed during the survey. Base stations need to be selected on the outer regions of the survey, so as to encompass the entire survey.

Preference must be given to stations which have a precise elevation referenced, by spirit level techniques, to the National Geodetic Vertical

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Point Positioning

$$D = \frac{2\sqrt{2} * \sigma_p}{1 / a}$$

σ_p = single coordinate standard deviation of Doppler point position (one sigma) in meters

a = denominator of distance accuracy classification standard (e.g. a = 100000, for 1:1000000 accuracy)

Order Class	First	Second I	Second II	Third I	Third II
precision, σ_p		Minimum distance (km)			
200 cm	566	242	114	56	28
100 cm	283	141	57	28	14
70 cm	200	100	40	20	10
50 cm	141	71	26	14	7

Datum (NGVD). This will allow geoidal height determinations to be made. At least two, preferably all, base stations shall be tied to the NGVD. It is preferable to have stations tied to the NGVD which span the largest portion of the survey. This allows an approximation of the geoidal slope to be made.

If none of the selected base stations are tied to the NGVD, at least two, preferably more, benchmark(s) of the National Vertical Network shall be occupied. Again, an attempt should be made to span the entire survey area.

Datum shifts for transformation of point position solutions will be obtained from the observations made on the base stations.

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Relative Positioning

$$D = \frac{2 * \sigma_r}{1 / a}$$

σ_r = single coordinate standard deviation of Doppler relative position (one sigma)

a = denominator of distance accuracy classification standard (e.g. a = 100000, for 1:100000 accuracy)

Order Class	First	Second I	Second II	Third I	Third II
precision, σ_r		Minimum distance (km)			
50 cm	100	50	20	10	5
35 cm	70	35	14	7	4
20 cm	40	20	8	4	2

Based upon the spacing of the Doppler stations and the desired order of the Doppler control, one can determine the required precision of the Doppler position(s) (σ_p or σ_r).

Instrumentation

The receivers must be of geodetic type and receive the two carrier frequencies transmitted by the NNSS. The receivers must record the Doppler count of the satellite, the receiver clock times, and the signal strength. The integration interval should be approximately 4.6 seconds. Typically 6 or 7 of these intervals are accumulated to form a 30-second Doppler count observation. The reference frequency must be stable to within 5.0 E-11 parts per 100 seconds. The maximum difference from the average receiver delay should not exceed 50 microseconds. The best estimate of the mean electrical center of the antenna should be marked. This mark will be the reference point for all height of antenna measurements.

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Calibration Procedures

Receivers should be calibrated at least once a year, or whenever a modification to the equipment is made. It is desirable to perform a calibration before every project to verify that the equipment is in an operational status. The two receiver method is prefered and should be used whenever possible.

Two Receiver Method

The observations are to be made on a three dimensional baseline, of high internal accuracy, 10 to 50 meters in length. The baseline should be located in an area free of radio interference in the 150 & 400 mHz frequencies. The 20 cm relative positioning field procedures will be used. The data is to be reduced with either short arc or semi-short arc methods. The receivers will be considered operational if the differences between the Doppler and the terrestrial baseline components do not exceed more than 40 cm (any coordinate axis).

Single Receiver Method

Observations will be made using the 50 cm field procedures, on a first order Doppler station. The data will be reduced using the precise ephemerides. The resultant position must agree within 1 meter of the established Doppler position.

One can establish their own calibration site, for future use, by first occupying a new, monumented station, followed by occupation of the established Doppler station. Again, 50 cm field procedures will be used, and the data reduced with the precise ephemerides. If the derived station position agrees with the established (1 meter), the position for the new station can be used for future calibrations.

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Field Procedures for Point and Relative positioning

Notice: the following tables of field procedures are valid only for measurements made with the Navy Navigation Satellite System (TRANSIT). The values for the precision estimates may not necessarily be applicable for surveys performed outside the CONUS.

Point Positioning

Point Precision, σ_p (1 sigma) (precise ephemerides)	50 cm	70 cm	100 cm	200 cm
Max. standard deviation of mean of counts/pass (cm), broadcast ephemerides	25	25	25	25
Period of observation not less than (hrs)	48	36	24	12
Number of observed passes not less than (1)	40	30	15	8
Minimum passes within each quadrant (2)	6	4	2	1
Number of acceptable passes (evaluated by on-site point position processing) not less than	30	20	9	4
Warm up time (hrs)				
crystal	48	48	24	24
atomic	1.5	1.5	1.0	1.0
Maximum interval between Meterological observations	6 hrs	(3)	(3)	(3)

(1) There should be a nearly equal number of north and south going passes
(2) Number of passes refers to passes for which the precise ephemerides are available for reduction
(3) Each set-up, take-down, and visit

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Relative positioning

Notice: Doppler station spacing must never exceed 500 km.

Relative Precision, σ_r (1 sigma)	20 cm	35 cm	50 cm
Maximum standard deviation of mean of counts/pass (cm), broadcast ephemerides	25	25	25
Period of observation not less than (hrs)	48	36	24
Number of observed passes not less than (1)	40	30	15
Minimum passes within each quadrant (2)	6	4	2
Number of acceptable passes (evaluated by on-site point position processing) not less than	30	20	9
Number of stations observed simultaneously	4	3	2
Warm-up time (hrs)			
crystal	48	48	48
atomic	1.5	1.5	1.5
Maximum interval between meterological observations	6 hrs	6 hrs	(3)

(1) Number of observed passes refers to all satellites available for tracking and reduction with the broadcast ephemerides
(2) The number of north and south going passes should be nearly equal
(3) Each set-up, take down, and visit

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Observation Procedures

The antenna must be located where minimum radio interference occurs (150 and 400 mHz frequencies). Medium frequency radar, high voltage power lines, transformers, excessive noise from automotive ignition systems, and high power radio and TV transmission antennas must be avoided. The horizon should not be obstructed above 7.5 degrees.

The antenna cannot be located near metal structures, or located less than two meters from the edge of a building when observing on a roof. The antenna must be stably located within 1 mm over the mark for the duration of the observations. The height difference between the station mark and the reference point for the antenna phase center shall be measured to the nearest millimeter. If an antenna is moved while a pass is in progress, that pass is not usable. Furthermore, the antenna must be relocated within 5mm of the original antenna height. If the antenna is not relocated to the stated value, the data must be processed as if two separate stations were established. In the case of a reoccupation of an existing Doppler station, the antenna should be relocated within 5mm of the original observing height.

Long-term reference frequency drift must be monitored to ensure it does not exceed the manufacturer's specifications.

The temperature and relative humidity should be collected, if possible, at or near the height of the phase center of the antenna. Observations of wet-bulb and dry-bulb temperature readings must be recorded to the nearest 0.5 degrees Centigrade. Barometric readings (station site pressure) must be recorded to the nearest 1.0 millibar and, if significant, they must be corrected for difference in height between the antenna and barometer. During automatic aquisition of Doppler data, continuous weather recording instruments can be used to collect meterological data.

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Office procedures

The processing constants and criteria for determining the quality of point and relative positioning results are as follows:

1. A data set should, on the average, have 20 Doppler counts per pass before processing.
2. The cut-off angle for both data points and passes will be 7.5 degrees.
3. The maximum allowable rejection of counts, 3 sigma post processing, will be 10 counts per pass.
4. The percent of data points rejected (excluding cut-off angle) for a solution should be less than 10 percent.
5. Depending on number of passes and quality of data, the standard deviation of the range residuals for all passes of a solution should range between:

Point Positioning	-	10 to 20 centimeters
Relative positioning	-	5 to 20 centimeters

A least squares adjustment, using arbitrary minimal constraints, will be checked for blunders by examination of the normalized residuals. The observation weights will be checked by examination of the post-adjustment estimate of the variance of unit weight. Distance standard errors computed by error propagation between points in a minimally constrained, correctly weighted, least squares adjustment will indicate the maximum achievable accuracy classification. The formula presented in the section on standards will be used to arrive at the actual classification. The least squares adjustment will use models which account for:

Tropospheric scale bias, 10% uncertainty
Receiver time delay
Satellite/receiver frequency offset
Precise ephemeris
Tropospheric refraction
Ionospheric refraction

A post least squares adjustment of the raw coordinate data may require models for the effect of long-term ephemeride variation and crustal motion on the adjusted results.

APPENDIX B.

IHO Special Publication No. 44

PART B - POSITIONS

Section B.1 - Horizontal control

B.1.1 - Primary shore control points should be located by survey methods at an accuracy of 1 part in 10 000. Where the survey is extensive, a higher degree of accuracy must be adopted to ensure that the relative positions are in error by not more than half the plottable error at the scale of the survey.

B.1.2 - When satellite positioning is used to determine the location of shore stations, ties should be made to the local horizontal datum.

B.1.3 - Where no geodetic control exists, a point of origin for the horizontal control should be determined by astronomical observations or satellite positioning, the probable error of which should not exceed 2" of arc or about 60 metres.

B.1.4 - Secondary stations, required for local positioning (usually visual) which will not be used for extending the control, should be located such that the error does not exceed the plottable error at the scale of the survey (normally 0.5 mm on paper).

B.1.5 - The position of soundings, dangers and all other significant features should be determined with an accuracy such that any probable error, measured relative to shore control, shall seldom exceed twice the minimum plottable error at the scale of the survey (normally 1.0 mm on paper). It is most desirable that whenever positions are determined by the intersection of lines of position, three such lines be used. The angle between any pair should not be less than 30°.

PARTIE B - POSITIONS

Section B.1 - Canevas géodésique

B.1.1 - La détermination des stations principales à terre devrait se faire par des méthodes de levés d'une précision de l'ordre de 1/10 000. Lorsque le levé est étendu, il s'avère nécessaire d'adopter un degré de précision supérieur afin d'assurer que l'erreur sur les positions relatives n'est pas supérieure à la moitié de l'erreur graphique à l'échelle du levé.

B.1.2 - Lorsque le positionnement par satellite est utilisé pour déterminer la position des stations à terre, des rattachements devraient être faits au système géodésique local.

B.1.3 - Là où il n'existe aucun canevas géodésique, un point d'origine du réseau géodésique devrait être déterminé à l'aide d'observations astronomiques ou d'un système de positionnement par satellite; l'erreur probable ne devrait pas, dans ce cas, être supérieure à 2" d'arc, soit environ 60 mètres.

B.1.4 - Les stations secondaires, nécessaires au positionnement local (généralement optique) qui ne seront pas utilisées pour l'extension du canevas géodésique, devraient être déterminées de manière à ce que l'erreur ne soit pas supérieure à l'erreur graphique à l'échelle du levé (normalement 0,5 mm sur le papier).

B.1.5 - La position des sondes, des dangers ou de tout autre élément significatif devrait être déterminée avec une précision telle que toute erreur probable, calculée par rapport aux stations du canevas géodésique à terre, n'excède qu'exceptionnellement deux fois l'erreur graphique minimum à l'échelle du levé (normalement 1,0 sur le papier). Il est très souhaitable que chaque fois que les positions sont déterminées par intersection de lignes de position, trois de ces lignes soient utilisées. L'angle formé par chaque paire de lignes

B.1.6 - The position of fixed navigational aids and offshore installations projecting above water should be determined, whenever practical, to the same standard as primary stations.

B.1.6 - La position des aides fixes à la navigation et des installations au large s'élevant au-dessus de la surface de l'eau devrait être déterminée, dans tous les cas où cela s'avère possible, selon les mêmes normes de précision que les stations principales.

B.1.7 - Floating aids to navigation should be fixed as precisely as practical and with a probable error not exceeding twice the minimum plottable error at the scale of the survey (normally 1.0 mm on paper).

B.1.7 - La position des aides flottantes à la navigation devrait être déterminée de manière aussi précise que possible et avec une erreur probable qui ne soit pas supérieure à deux fois l'erreur graphique minimum à l'échelle du levé (normalement 1,0 mm sur le papier).

PART C - DEPTHS

PARTIE C - PROFONDEURS

Section C.1 - Measured depths

Section C.1 - Profondeurs mesurées

C.1.1 - The error in measuring the depths should not exceed :

(a) 0.3 metre from 0 to 30 metres
(b) 1.0 metre from 30 to 100 metres
(c) 1% of depths greater than 100 metres.

C.1.1 - L'erreur dans la mesure de la profondeur ne devrait pas être supérieure à :

(a) 0,3 mètre, de 0 à 30 mètres
(b) 1,0 mètre, de 30 à 100 mètres
(c) 1% des profondeurs supérieures à 100 mètres.

C.1.2 - Measured depths must be reduced to the sounding datum by application of the tidal height. The error of such reductions should not exceed the errors acceptable for depth measurement specified in C.1.1. Depths greater than 200 metres normally need not be reduced for tidal height.

C.1.2 - Les profondeurs mesurées doivent être rapportées au niveau de référence par déduction de la hauteur de la marée. L'erreur sur de telles réductions ne devrait pas être supérieure à l'erreur acceptable pour la mesure des profondeurs figurant au point C.1.1. Normalement, il n'est pas nécessaire d'appliquer la réduction de marée aux profondeurs supérieures à 200 mètres.

C.1.3 - A difference in depth at the intersection of two crossing lines of soundings which exceeds twice the relevant values given in C.1.1 should be investigated. Such a discrepancy may be due to an error in position, sounding or tidal reduction.

C.1.3 - Toute différence de profondeur à l'intersection de deux profils de sonde traversiers qui dépasserait le double des valeurs pertinentes figurant au point C.1.1 devrait faire l'objet de vérification. Une telle différence peut être due à une erreur de position, de sonde ou de réduction de marée.

APPENDIX C

GEODOP V REDUCTION PROCEDURES

The following is a brief description of the procedures and options used at NGS for processing Doppler data (MX-1502) with GEODOP V. These procedures were defined based on test runs with data observed on a high precision standard (Transcontinental Traverse (TCT)) and conversations with Mr. Jan Kouba of the Geodetic Survey of Canada. The procedures are specified with the understanding that less than maximum accuracy may result due to the use of a "standard" procedure.

This paper is meant only to enhance some sections of the GEODOP User's Guide written by J. Kouba, references to that paper are indicated by [KOUBA]. It is highly recommended that the User's Guide be consulted first; this paper deals only with methods at NGS and does not present any alternative methods for performing data reductions.

It is assumed that each station data set was observed with only one receiver. If this is not the case, the data set must be sub-divided into single receiver data sets. The station data set must also be sub-divided if the antenna was not re-established within ± .005m even if the same receiver was used to perform both occupations. The NGS version of GEODOP has been modified to accept antenna height to the nearest centimeter; the standard version reads the height to the nearest decimeter.

Because NGS also performs reductions with the precise ephemeris (program DOPPLR) the station coordinates are well known at the beginning of the GEODOP reduction. Use of these coordinates, in both PREDOP and GEODOP, reduces the number of runs which must be performed. At present, precise ephemeris reductions are not performed on a production basis at NGS.

The first step in performing a multi-station reduction is to first reduce all station data with single station reductions. These runs are performed primarily to obtain an estimate of the range rate sigma (RRS) for each station. Improved estimates of FRCV, SIGF, and SIGC, also are obtained. If a user does not have a good estimate of the station coordinates, these runs can also be used to refine the approximation. These updated coordinates can then be used in a second PREDOP run [KOUBA] and subsequent GEODOP runs.

RRS

To obtain an improved estimate of the range rate sigma (RRS), the RRS used for the single station reduction should be multiplied by the SO (estimated standard deviation of unit weight) of the reduction. The default value of 15 cm is used for the single station reduction. The improved estimate will be used as the initial RRS in the multi-station reductions.

NDLY

The receiver time delay (NDLY) for each receiver should be used. This value can be either directly measured in a lab or can be computed (as is done at NGS) during a DOPPLR reduction. If the value is known for one of the receivers in the survey, GEODOP V can be used to compute the other receiver delays [KOUBA]. The approximate value for the MX-1502 is 200 to 300 micro sec.; for geoceivers the delay is 1000 to 1100 micro sec.. These values are both receiver and manufacturer specific. Approximate values for other receivers can be obtained from the manufacturer.

RT

For optimum results the correlation model (RT) should be 2. With this option a standard deviation and correlation are computed for each pass. This option had the single most effect on the test reductions performed at NGS. As stated in [KOUBA] the option is "expensive" in that the computation time is nearly doubled.

MSTA

The number of simultaneous passes switch (MSTA) should be set to the number of active receivers used in the survey. If the solution will entail more stations than receivers, (ie receivers were moved about during the survey) it may be adviseable to set MSTA to one less than the number of receivers. This is suggested since the current version of GEODOP will terminate after encountering 10 passes where there are not enough simultaneous observations. This termination does not produce an error message.

Data set size should be limited to no more than 5 to 10 days. Due to variations in the broadcast ephemeris, larger data sets will cause degradation of the solution. Large surveys, spanning longer periods, should be reduced in segments with these segments being joined with an adjustment program such as NASSTI or GLDSAT.

NORB

The number of orbital biases (NORB) computed is normally set at 4 on surveys with baselines less than 500 km.

STWGHT

The orbital constraints (STWGHT(1-6)) can usually be lowered from the default value of 10 m. If the individual orbital biases are generally below 15 m (±) the appropriate constraint(s) can be lowered to 5 m. Currently at NGS, all but STWGHT(2) have been set to 5 m. STWGHT(2) is set to 7.5 m. Since these values are based on the means of the orbital biases, a change in the program code has been planned (at NGS) which would provide the mean of the biases as part of the GEODOP output.

SO

The estimated standard deviation of unit weight is output for both the entire reduction and for each station which was used in the reduction. The individual station SO should be between .90 and 1.00. The spread between all of the station SOs should be less than .10. Reductions performed with the TCT data indicate that the most accurate baseline determinations are made when the station SOs are near .95, with a spread of less than .05 .

If a single station SO varies from the others by more than .10 the station values should be inspected. Specifically, the RRS and NDLY values should be verified or changed. Assuming the NDLY for the station is correct, the RRS value can be changed to improve the station SO. A low SO indicates that the RRS estimate was too pessimistic and that it should be lowered.

Too high indicates the RRS was optimistic and should be increased. Experience with the MX-1502 shows this value is usually 10 cm or lower. If need be, each station SO can be changed to optimize the solution (solution SO = .95, with minimum spread between station SOs).

FOFFS

Generally, frequency drift and offset can be left at the default values.

ICPA

Trimming of pass data about the CPA (ICPA=1) may improve the solution results. Limited testing at NGS has shown a slight degradation of solution quality, but this may not be the general case. CPA trimming might be "dangerous" at stations where the horizon is obstructed in a quadrant. Trimming could cause rejection of low elevation data points, resulting in a poor solution.

POSRED

The NGS version of POSRED has been modified to compute the standard deviation of the position differences. These sigmas are used to estimate the relative accuracy of the station positions. Because these values are computed from the variance-covariance and correlation matrices they are affected by the weighting and a-priori values used in the reduction. When using these sigmas for comparison purposes, one should verify that the solution SO is near 1.00 . The closer the SO is to 1.00 the more realistic the sigmas.

SIGA

The a priori variance (SIGA) value used is 1.0 (default is 1.4).

Precise Ephemeris Reduction

Limited testing with precise ephemeris reductions have been very promising. Using program NMERGE station files were created with the broadcast ephemerides replaced by the precise ephemerides. NMERGE must be run once for each satellitte for which there is precise ephemeris. Point position results agreed well with program DOPPLR (NGS-03) reductions. Multi-station reductions also agreed well between precise and broadcast ephemeris reductions. On a 92 km baseline, the baseline lengths, determined with GEODOP V using both ephemerides, agreed to 4 centimeters. When performing GEODOP V runs with the precise ephemerides the orbital biases (STWGHT(1-6)) are set to .01 m. The number of biases used (NORB) is 6.

Attached are option cards from reductions performed on the TCT Doppler data. The TCT measurements were used as the standard to which the Doppler data were compared for baseline accuracy.

PREDOP OPTION CARDS

```
         1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
         !         !         !         !         !         !         !         !

  OBS RM 1    50231 39. 08. 11.496282. 48. 04.380      114.9  0.0  0.0  0.0
5  0.  0.0    1  5.0 10.01000.  50.     1 1450 7598    0    0              33 115.

         !         !         !         !         !         !         !         !
12345678901234567890123456789012345678901234567890123456789012345678901234567890
         1         2         3         4         5         6         7         8

      GEOS    50222 39. 01. 15.293283. 10. 20.240       13.4  0.0  0.0  0.0
5  0.  0.0    1  5.0 10.01000.  50.     1 1450 7598    0    0              33 115.

   HERNDON    30691 38. 59. 43.223282. 41. 11.245       75.3  0.0  0.0  0.0
5  0.  0.0    1  5.0 10.01000.  50.     1 1450 7598    0    0              33 115.
```

GEODOP SINGLE STATION RUN OPTION CARDS

```
         1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
         !         !         !         !         !         !         !         !

  01   01    4   32    4 5 50  10. 7660-4458 81641792. 2980 2351 129. 1982 1982
 5  0   75    5  14 145. 75977 0.   0.   0.   10.   1.     010.10.10.10.10.10.00
         1
    50231 0 2.15  318                 0.   0.  5.76 10976251-48307402 40041616

         1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
         !         !         !         !         !         !         !         !

  01   01    4   32    4 5 50  10. 7660-4458 81641792. 2980 2351 129. 1982 1982
 5  0   75    5  14 145. 75977 0.   0.   0.   10.   1.     010.10.10.10.10.10.00
         1
    50222 0 2.15  402                 0.   0.  1.44 11307137-48313317 39941341

         !         !         !         !         !         !         !         !
12345678901234567890123456789012345678901234567890123456789012345678901234567890
         1         2         3         4         5         6         7         8

  01   01    4   32    4 5 50  10. 7660-4458 81641792. 2980 2351 129. 1982 1982
 5  0   75    5  14 145. 75977 0.   0.   0.   10.   1.     010.10.10.10.10.10.00
         1
    30691 0 2.15  254                 0.   0.  1.46 10901099-48425360 39919668
```

GEODOP 3 STATION REDUCTION OPTION CARDS

```
         1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
         !         !         !         !         !         !         !         !

  03   03    4   32    4 5 50  10. 7660-4458 81641792. 2980 2351 129. 1982 1982
 5  0   75   35  10 145. 75977 0.   0.   0.   10.   2.     005.7.505.05.05.05.00
         3         0         0         0         0         0         0 0.
    50222 0 2.09  402          00     0.   0.  1.44 11307137-48313317 39941341
    50231 0 2.10  318          00     0.   0.  5.76 10976251-48307402 40041616
    30691 0 2.10  254          00     0.   0.  1.46 10901099-48425360 39919668
```

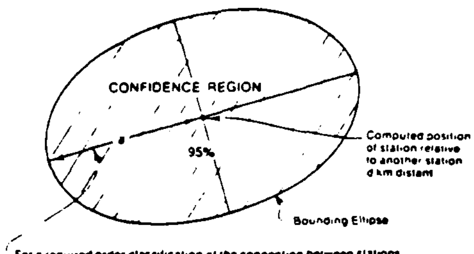
APPENDIX D.

Specifications of the Geodetic Survey of Canada

PART 2 — HORIZONTAL CONTROL

INTRODUCTION

In August, 1973, the Surveys and Mapping Branch published "Specifications and Recommendations for Control Surveys and Survey Markers". Those specifications were specifically designed for the most common types of control surveys carried out by the Branch and did not contain specific provision for short lines. As a result of greater interest in urban control surveys, the Branch prepared "Specifications and Recommendations for Horizontal Control Surveys with Short Lines" in June 1975, as a provisional supplement to the 1973 publication. These specifications combine the 1973 and 1975 specifications.



For a required order classification of the connection between stations the semi-major axis a must be less than r=C (d+0.2) cm where C has the value assigned for the order

Figure 1

Ellipse Showing the 95% Confidence Region of One Station Relative to Another (the area within which there is a 95 percent probability of the true relative position being situated).

SPECIFICATIONS

Horizontal control surveys are classified as first, second, third or fourth-order according to standards of accuracy.

The statistical concepts of standard deviation and confidence region are used to define standards of accuracy. These statistical concepts replace the concept of maximum anticipated error used in the Branch specifications issued in 1961 (See Appendices A and B).

A survey station of a network is classified according to whether the semi-major axis of the 95 percent confidence region, with respect to other stations of the network, is less than or equal to: $r = C\,(d+0.2)$, where r is in centimetres, d is distance in kilometres to any station, and C is a factor assigned according to the order of survey. An ellipse bounding the 95 percent confidence region is shown in Figure 1. For first-order, the value assigned to C is 2. This means that for a station to be classified as first-order, the semi-major axis of the 95 percent confidence region must be less than or equal to $r = 2d + 0.4$.

For two stations 10 km apart, $r = 20.4$ cm. For these stations to be classified as first-order, the semi-major axis of the 95 percent confidence region of one station relative to the other must be less than or equal to 20.4 cm. The values of C assigned to various orders of survey are shown in Table I (Figure 2 is a graph of r against distance. See also Table II).

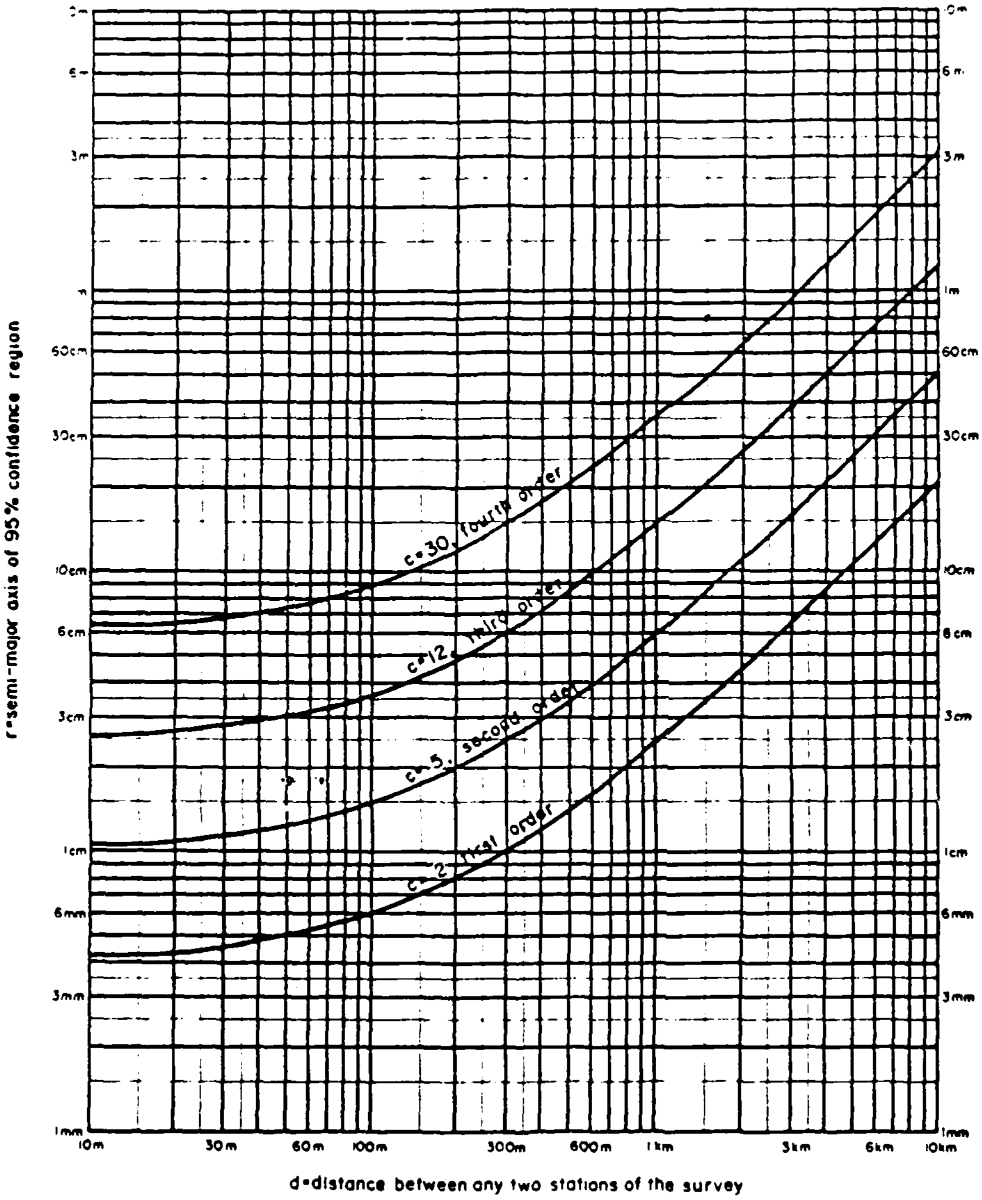
TABLE I

VALUES OF C FOR HORIZONTAL CONTROL SURVEYS ACCORDING TO ORDER, USING $r = C(d+0.2)$
(r is in cm, d is distance in km)

ORDER	C
1st	2
2nd	5
3rd	12
4th	30

As noted in Table II, the use of $r = C\,(d+0.2)$ causes the parts per million (ppm) and ratio values to change significantly with distance, for short lines; this reflects practical considerations. Experience shows that with most modern methods of establishing closely-spaced control, the overall pattern of error propagation — the combination of instrumental and centering errors, the effects of network configuration, and a host of other contributing errors, most of which defy individual identification — is not proportional to distance.

The errors of measurement contributing to this pattern can be divided into two groups; those proportional to distance and those that are independent of distance. As lines become shorter, the second group becomes dominant. For the commonly used short-distance measuring instruments, the first group is dominant above three kilometres, and the second group is significant within the range zero to three kilometres. Therefore, these specifications are useful for surveys with points either closely or widely spaced or with a mixture of both



NOTE: this graph is logarithmic

Figure 2

Accuracy Standards for Horizontal Control Surveys

(based on r = C(d + 0.2) [illegible]

Where a survey network is distorted by constraint inaccuracies in positions held fixed, examination of the adjustment results should be made beyond merely observing whether the error ellipses are within these accuracy standards. This examination should include a study of the residuals and the relative shift in positions between free and constrained adjustments. In computing standard error ellipses for networks under constraint, the computed standard deviation for unit weight from the adjustment should be used. Sometimes this means that stations, which would be classified as a first-order survey by an unconstrained adjustment, must be classified as lower-order until a general readjustment removes the distortion.

Guidelines on network design and measurements are given in "Network Design" to assist in achieving the various orders of accuracy. However, *it is stressed that by merely following the guidelines one does not ensure the achievement of the order of accuracy desired.* The order can *only* be confirmed by an analysis of the survey results.

NETWORK DESIGN

The size and shape of the confidence region is dependent not only on the accuracy of the field measurements but also on the configuration of the control network.

For a network to fulfill its basic role as a strong and reliable reference framework, it must be homogeneous, feature a reasonable number of redundancies, and the individual figures should be well-shaped. Stations should be as evenly spaced as possible, and all adjacent pairs of stations in the network should preferably be connnected by direct measurement. The ratio of the longest length to the shortest should never be greater than five and usually should be much less.

A basic principle of control surveys is to work from the large to the small, therefore, the spacing of higher-order control stations should generally be greater than that of lower-order stations. In addition, there should always be a sufficient density of higher-order control to govern the establishment of lower orders.

Frequently, these ideals cannot be realized. Reality is often a network that has adjacent points which cannot be conveniently connected, that has large variation in lengths, and that has been measured with various instruments with significantly different accuracies. The surveyor must design the network with these factors in mind.

To design a network to achieve required accuracies, good *a priori* estimates of the accuracies of various instruments used with various techniques must be available. *These estimates must reflect not only the consistency of* several measurements of the same quantity by the same instrument, over a short interval of time under ideal conditions, but must also reflect normal random errors *likely to occur in normal field use, under normal operating* conditions by personnel who take only normal precautions. In addition, the estimates must take into account systematic errors that may not be evident *in a normal* survey; for example, an uncorrected zero error in Electronic Distance Measuring (EDM) instruments, systematic meteorological errors due to imperfect measuring techniques, etc. Appendix E lists typical standard deviations that may be expected under normal circumstances and which may be used to compute weights in network design programs. Higher accuracies should be estimated if extraordinary precautions are taken in calibration and measurement.

The accuracy of a horizontal control survey can be assessed properly from the results of a rigorous least-squares adjustment of the measurements. Since this assessment can only be made after the field work has been completed, something more helpful is needed for those who wish to design networks and prepare measurement guidelines, and who require some reasonable assurance that a particular order of accuracy will be obtained when the field work is done.

TABLE II

ACCURACY STANDARDS FOR HORIZONTAL CONTROL SURVEYS

(showing the variation in proportional accuracy over short distances)

ORDER	C	SEMI-MAJOR AXIS OF 95% CONFIDENCE REGION, $r = C(d + 0.2)$, WHERE d IS THE DISTANCE BETWEEN ANY TWO STATIONS																	
		for d = 0.03 km			for d = 0.1 km			for d = 0.3 km			for d = 1.0 km			for d = 3.0 km			for d = 10 km		
		cm	ppm	ratio	cm	ppm	ratio	cm	ppm	ratio	cm	ppm	ratio	cm	ppm	ratio	cm	ppm	ratio
1	2	0.5	153	1/6500	0.6	60	1/16700	1.0	33	1/30000	2.4	24	1/41700	6.4	21	1/46900	20	20	1/50000
2	5	1.2	383	1/2600	1.5	150	1/6700	2.5	83	1/12000	6.0	60	1/16700	16.0	53	1/18800	50	50	1/20000
3	12	2.8	920	1/1100	3.6	360	1/2800	6.0	200	1/5000	14.4	144	1/6900	38.4	128	1/7800	120	120	1/8300
4	30	6.9	2300	1/430	9.0	300	1/1100	15.0	500	1/2000	36.0	360	1/2800	96.0	[illegible]	1/3100	300	[illegible]	[illegible]

The best course of action is to simulate the proposed network in a suitable computer program such as GALS* using *a priori* estimates for the standard deviations of the proposed measurements (see Appendix E). The results of such a simulation study, tempered with the wisdom of practical experience, usually provide a reliable indication of the accuracy likely to be obtained in the field.

For those not able to conduct computer simulation studies, some aids are provided in this publication.

- Appendix C provides measurement guidelines for the conventional methods — triangulation, traversing and *trilateration* — based on practical experience, and the results of computer simulation studies of simple idealized networks. *At best, these guidelines are a general guide only and must be treated with caution.* The reader should pay particular attention to the characteristics of the idealized networks depicted therein, to determine whether extrapolation can reasonably be made from the guidelines to the task at hand. The reader should also pay attention to the explanatory notes which follow the network sketches in Appendix C.
- Appendix D demonstrates some simple calculations that can be of benefit in estimating the accuracy of points in a network.
- Appendix E lists typical standard deviations, stemming from practical experience, for distances, directions, azimuths and position differences measured using various instruments and methods of observation.

PHOTOGRAMMETRIC METHODS

On occasion, horizontal control can be densified effectively using photogrammetric methods (see Appendix F).

*A Geodetic Survey [illegible]

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