Doc 9674
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World Geodetic System — 1984 (WGS-84) Manual

Approved by the Secretary General
and published under his authority

Second Edition — 2002

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AMENDMENTS

The issue of amendments is announced regularly in the *ICAO Journal* and in the monthly *Supplement to the Catalogue of ICAO Publications and Audio-visual Training Aids*, which holders of this publication should consult. The space below is provided to keep a record of such amendments.

RECORD OF AMENDMENTS AND CORRIGENDA

AMENDMENTS		
No.	Date	Entered by

CORRIGENDA		
No.	Date	Entered by

FOREWORD

The Council of the International Civil Aviation Organization (ICAO), at the thirteenth meeting of its 126th Session on 3 March 1989, approved Recommendation 3.2/1 of the fourth meeting of the Special Committee on Future Air Navigation Systems (FANS/4) concerning the adoption of the World Geodetic System — 1984 (WGS-84) as the standard geodetic reference system for future navigation with respect to international civil aviation. FANS/4 Recommendation 3.2/1 reads:

> "**Recommendation.— Adoption of WGS-84**
>
> That ICAO adopts, as a standard, the geodetic reference system WGS-84 and develops appropriate ICAO material, particularly in respect of Annexes 4 and 15, in order to ensure a rapid and comprehensive implementation of the WGS-84 geodetic reference system."

The Council, at the ninth meeting of its 141st Session on 28 February 1994, adopted Amendment 28 to Annex 15, introducing the provisions concerning the promulgation of WGS-84-related geographical coordinates. Consequential amendments to Annexes 4, 11 and 14, Volumes I and II, were adopted by the Council on 1 March 1995, 18 March 1994 and 13 March 1995, respectively. On 20 March 1997 the Council, at the seventeenth meeting of its 150th Session, adopted Amendment 29 to Annex 15 introducing publication of the vertical component of the WGS-84 geodetic reference system. Consequential amendments to Annexes 4 and 14, Volumes I and II, were adopted by the Council on 20 March 1998 and 21 March 1997, respectively. The Standards and Recommended Practices (SARPs) in Annexes 11 and 14, Volumes I and II, govern the determination (accuracy of the field work) and reporting of geographic coordinates in terms of the WGS-84 geodetic reference system. Annexes 4 and 15 SARPs govern the publication in textual or graphical form of geographic coordinates (resolution) and vertical components. States' aeronautical information service departments will publish in their Aeronautical Information Publications (AIPs), on charts and store in electronic databases where applicable, geographic coordinate and vertical component values based on WGS-84 which are supplied by the other State aeronautical services, such as the air traffic service and the aerodrome/heliport authority.

The purpose of this manual is to furnish guidance on the provision of geographic coordinates and vertical component values referenced to the WGS-84 datum in order to assist States in the uniform implementation of the SARPs on WGS-84 as contained in:

Annex 4 — *Aeronautical Charts*;

Annex 11 — *Air Traffic Services*;

Annex 14 — *Aerodromes*,
Volume I — *Aerodrome Design and Operations* and
Volume II — *Heliports*;

Annex 15 — *Aeronautical Information Services*.

This manual has been prepared in consultation and coordination with the European Organization for the Safety of Air Navigation (EUROCONTROL) and will be amended periodically. Users are invited to forward to ICAO suggestions for improvements or additions based on their practical experience when using the manual. Errors or discrepancies noticed in the manual should be brought to the attention of:

The Secretary General
International Civil Aviation Organization
999 University Street
Montreal, Quebec, Canada H3C 5H7

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Chapter 1

INTRODUCTION

1.1 EFFECTS OF USING DIFFERING COORDINATE REFERENCE SYSTEMS IN AVIATION

1.1.1 Geodetic datum problems in air navigation were first encountered in Europe in the early 1970s during the development of multi-radar tracking systems for EUROCONTROL's Maastricht Upper Airspace Centre (UAC), where plot data from radars located in Belgium, Germany and the Netherlands were processed to form a composite track display for air traffic controllers. Discrepancies in the radar tracks were found to be the result of incompatible coordinates.

1.1.2 In the mid-1970s, during trajectography experiments with the French SAVVAN system (Système Automatique de Vérification en Vol des Aides a la Navigation, i.e. Automatic In-flight Navigation Aids Checking System), positional "jumps" were noticed when switching between Distance Measurement Equipment (DME) transponders located in different States. Once more, the errors could only be attributed to incompatibility of the coordinates of ground aids.

1.1.3 If a ground-based radar navigation aid is coordinated in two or more different geodetic reference datums, aircraft horizontal position determination will have two or more different sets of latitude and longitude values. In metric units aircraft locations could show a discrepancy of up to several hundred metres when simultaneously located and tracked by two radars: Radar datum 1 and Radar datum 2 (see Figure 1-1). This could lead to a situation where an aircraft close to a border between two States with different geodetic reference datums could be seen by radars in the two States as having different positions and the potential for misinterpreting aircraft separations and clearances from restricted areas.

1.1.4 Thus, the main source of systematic errors is the non-use of a common geodetic reference datum for determining radar positions; the solution is to derive the radar positions from a common geodetic reference frame.

1.1.5 There are many geodetic reference datums in use throughout the world providing references for the charting of particular areas. Each datum has been produced by fitting a particular mathematical earth model (ellipsoid) to the true shape of the earth (geoid) in such a way as to minimize the differences between the ellipsoid and the geoid over the area of interest. Most ellipsoids in current use were derived in the 1800s and were normally referenced to a local observatory. These different datums and ellipsoids produce different latitude and longitude grids and, hence, different sets of geographical coordinates. States developed their own geodetic datums which usually differed from those of adjacent States. As distance requirements increased beyond national boundaries, new requirements arose for datums on at least a continental scale.

1.1.6 Looking at the current situation, it must be acknowledged that in the en-route environment, the use of ground-based navigation aids in different reference frames does not have any significant effect since the primary means of navigation remains the use of VOR or NDB signals to define radial tracks to or from the beacon with turning points either at the beacon or at a distance from it, defined by the DME. In such circumstances, published coordinates of the navaid do not affect the track flown by the aircraft. This will dramatically change either in the approach and landing phase or where reduced lateral aircraft separation is implemented, i.e. Area Navigation (RNAV) and Required Navigation Performance (RNP) systems with higher accuracy and integrity requirements. Therefore, these discrepancies will no longer be tolerable and will demand the introduction of a common geodetic reference system for use by international civil aviation.

1.1.7 The United States Department of Defense (World Geodetic System Committee) defined and developed a number of geocentric reference systems to which other geodetic networks may be referred. The continued development using increasingly available satellite information resulted in the World Geodetic System — 1960 (WGS-60), — 1966 (WGS-66), — 1972 (WGS-72) and the current definition — 1984 (WGS-84).

1.2 MAGNITUDE OF THE PROBLEM

1.2.1 The discrepancies between one geodetic reference frame and another depend upon the:

a) order of magnitude of the three origin shifts;

b) magnitude of the three axial rotations;

c) scale factor value; and

d) shape of the reference ellipsoid (if working in geographical coordinates).

Note.— Most States already have a national reference frame with a specific set of datum parameters. The datum discrepancies range from metres to kilometres.

1.2.2 Figure 1-2 illustrates the magnitude of positional differences for Europe between points expressed in different geodetic datums. The figure represents the differences in seconds of arc between values in national geodetic datums and WGS-72 in five States for latitude and longitude, respectively. WGS-72 has been used for this illustration because the transformation parameters from the national geodetic datums were known. From Figure 1-2, it can be deduced that the differences in position of points with respect to different national geodetic datums and WGS-72 can be in the order of a few hundred metres for a particular State.

1.3 NAVIGATIONAL IMPLICATIONS

1.3.1 Geographical coordinates used in the civil aviation environment today are generally of two type: ground-derived coordinates and navigation system-derived coordinates. Ground-derived coordinates are those that are obtained through surveys, calculations and measurements. They are published by the civil aviation authorities in Aeronautical Information Publications (AIPs) and charts made available to the public. Navigation system-derived coordinates, on the other hand, are coordinates generated by the airborne systems from accelerometers and ground-based or satellite-based signals.

1.3.2 Ground-derived coordinates (latitude and longitude) are determined with measurements and calculations on mathematical reference models. These models represent the shape of the earth in a particular geographic region and are called geodetic datums. For example, coordinates used by civil aviation in the United States are mathematically referenced or calculated to the North American Datum (NAD), in Japan to the Tokyo Datum (TD) and in Europe to the European Datum (ED). Each of these datums uses a different mathematical model that “best fits” or provides the best representation of the earth’s shape in that specific geographic region. Even though States seldom publicize a geodetic datum, it is common practice for a State to use a specific datum for all mapping, charting and geodetic activities. The mathematical parameters of these datums differ, the location of the centre of each datum differs and, except for those States that have already converted to an earth-centred datum, none of the datum centres coincides with the centre of gravity of the earth.

1.3.3 Unlike ground-derived coordinates, navigation system-derived coordinates are earth-centred. The Inertial Navigation System (INS) uses accelerometers on a gyro- or laser ring-stabilized platform to sense movement and determine aircraft position. The alignment of the platform relates to the earth’s centre of mass and rotation resulting in INS-generated coordinates that are referenced to the earth’s centre. This means that published coordinates as referenced to local geodetic datums will not compare, directly, with INS-generated coordinates. Because INS is normally aligned with local coordinates before take-off, it is the most accurate within the area defined by the local datum. Inter-datum flights up to the present have not been hindered by the “coordinate shift” which is small compared with the drift of the INS on the en-route phase of long-distance flights.

1.3.4 Coordinates derived by the airborne Global Navigation Satellite System (GNSS) from signals received from satellites will be earth-centred because the GNSS satellites operate with an earth-centred reference model, i.e. WGS-84. GNSS coordinates will not compare with coordinates based on local geodetic datums except in areas where coordinates have been readjusted to an earth-centred datum. This means that the difference between the coordinates of a point referenced to a local geodetic datum and the coordinates of that same point referenced to the earth-centred WGS-84 datum has to be taken into account.

1.4 SOLUTION TO THE PROBLEM

1.4.1 The solution to this problem was to adopt WGS-84 as a common geodetic reference frame for civil aviation (see the Foreword). To facilitate implementation of the WGS-84 reference frame, this guidance material was prepared.

1.4.2 The first step in the implementation of any coordinate transformation proposal is to carry out an inventory. In order to make an assessment of the quality of the published aeronautical geographical coordinates required for air navigation, it is necessary to review all existing related records of aeronautical coordinate data.

1.4.3 A sample questionnaire designed for survey inventory is provided in Appendix G. Information provided through the use of such a questionnaire will allow for accurate estimates and identification of those items for which a field survey is required in order to verify positions.

1.4.4 Analysis of the questionnaire data will identify the navigation aids and aerodrome/heliport points and facilities that need to be resurveyed. It will also identify those positions where the geographical coordinates satisfy the required accuracy and integrity to allow for direct transformation to the WGS-84 geodetic reference frame by mathematical means alone.

1.4.5 In principle, there are two approaches which can be used as stand-alone or combined methods to transform a survey given in adequately accurate coordinates to WGS-84.

a) Survey at least three control stations (covering the area under consideration) to obtain WGS-84 coordinates and determine the datum parameters between the local reference frame and WGS-84.

b) Determine by a computational datum transformation WGS-84 coordinates for all remaining points.

1.4.6 There are two general groups of air navigation points for which geographical coordinates are required (see Table 1-1).

1.4.7 The preceding paragraphs considered the horizontal element of the WGS-84 geodetic system. However, WGS-84 is a three-dimensional reference frame coordinated in X, Y, Z or in φ, λ and h. Geographical coordinates are expressed in latitude φ and longitude λ while the parameter h is the geometric (ellipsoidal) height above the WGS-84 ellipsoid.

1.4.8 GNSS-derived heights are referenced to the WGS-84 ellipsoid which will usually differ from the "normal" (orthometric) height at the same point. The difference will be of significance in the aerodrome environment when navigating with GNSS sensors. The difference between orthometric height (geoid height, elevation) and WGS-84 ellipsoidal height must therefore be made available to the aviation community. The height that separates geoid and WGS-84 ellipsoid is the geoid undulation.

1.4.9 Geoid undulation is required for airport elevations, runway thresholds and touchdown and lift-off areas (TLOFs) or thresholds of final approach and take-off areas (FATOs) at heliports. (See also Appendix B.)

Table 1-1. Air navigation-related coordinates of interest

Area/en-route coordinates	*Aerodrome/heliport coordinates*
ATS/RNAV route points	Aerodrome/heliport reference points
Holding points	Runway, FATO thresholds
En-route radio navigation aids	Terminal radio navigation aids
Restricted/prohibited/danger areas	FAF, FAP and other IAP essential points
Obstacles — en route	Runway centre line points
FIR boundaries	Aircraft standpoints
CTA, CTZ	Aerodrome/heliport obstacles
Other significant points	

FIGURES FOR CHAPTER 1

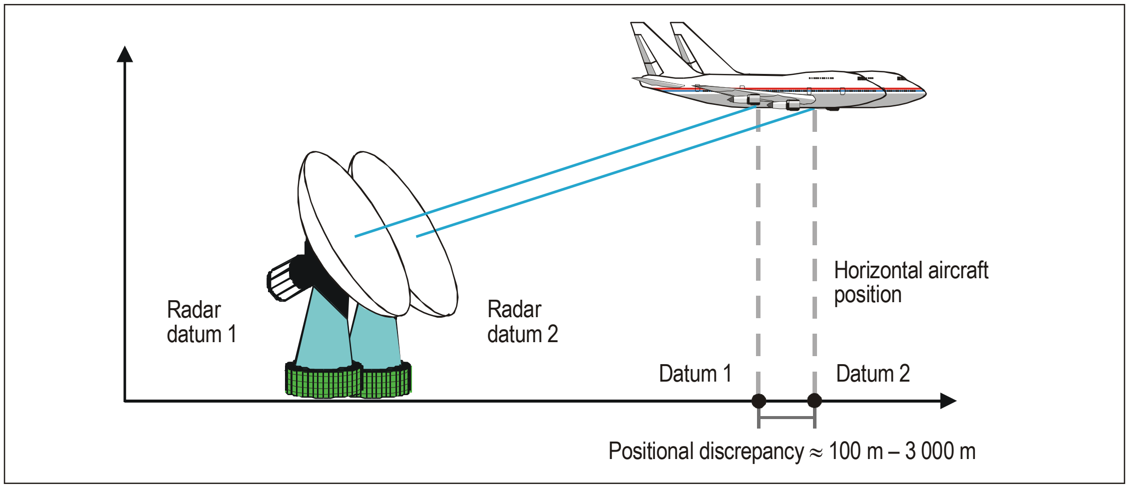

Figure 1-1. Datum problem in air navigation

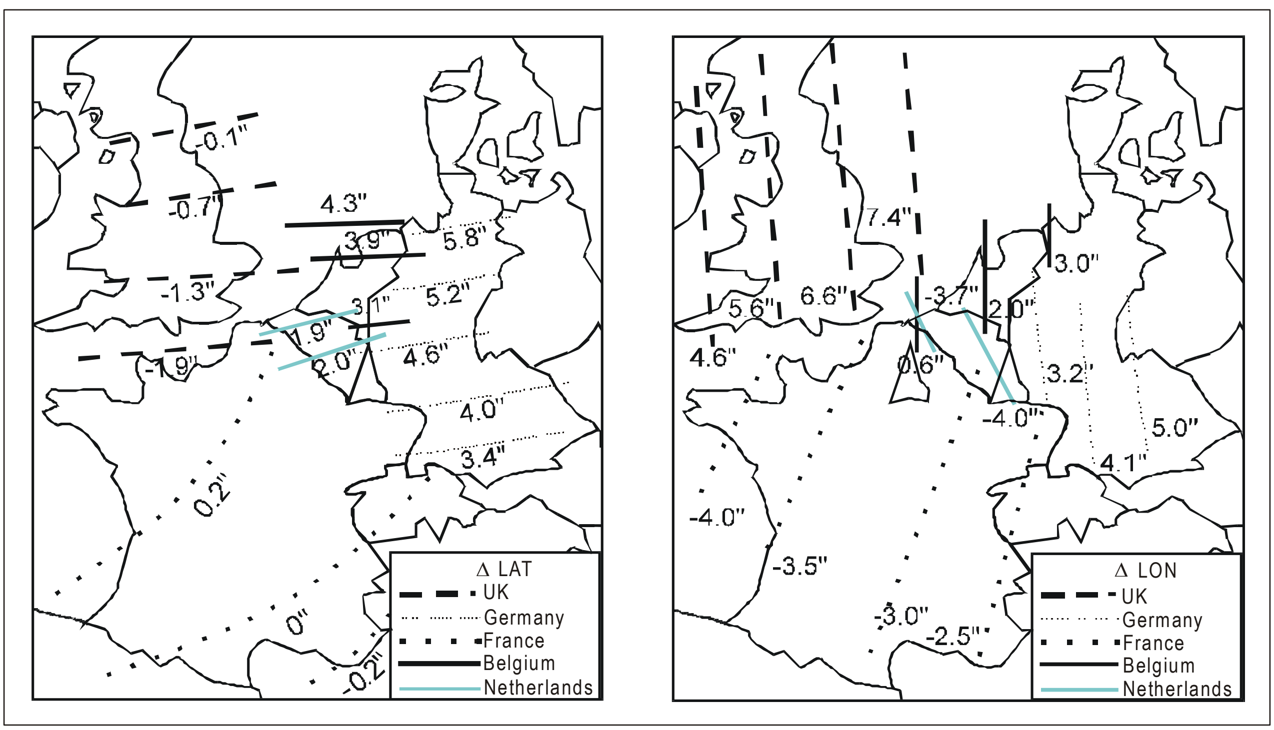

Figure 1-2. ΔLAT, ΔLON between local and WGS-72 (")

Chapter 2

ACCURACY, RESOLUTION AND INTEGRITY OF AERONAUTICAL DATA

2.1 GENERAL

2.1.1 Traditional navigation techniques have relied upon the ability to fly to or from point navigation aids. While the coordinates of the navigation aids have been provided, this information has not been used as part of the navigation process. Increasing use is being made of Area Navigation (RNAV) systems which derive the aircraft position from such sources as Inertial Navigation Systems (INS), Omega, VHF omni-directional radio range/distance measuring equipment (VOR/DME), dual or multi-DME and Global Navigation Satellite Systems (GNSS). Based on aeronautical data, RNAV systems generate appropriate instructions to the autopilots which enable the aircraft to follow the planned route during the departure, en-route and approach phases and eventually, with the implementation of GNSS, the landing phases.

2.1.2 For such operations, the track actually flown by the aircraft depends upon the coordinates defining both the track and the position of ground-based navigation aids. With the advent of precision RNAV (RNP 1) routes and the extension of RNAV application to terminal area (TMA) procedures, higher precision is required, and it is necessary to ensure that the data defining the track to be flown are of an accuracy, resolution and integrity consistent with the RNP requirements.

2.2 TYPE AND CLASSIFICATION OF POSITIONAL DATA

2.2.1 Air navigation points can be divided into two basic groups (as outlined in Table 1-1):

a) area/en-route points; and

b) aerodrome/heliport points.

2.2.2 Besides this basic categorization, air navigation points can be categorized by the type of positional data. Three types of positional data have been defined: surveyed points, calculated points and declared points (see Tables 2-1 to 2-5).

a) *Surveyed point.* A surveyed point is a clearly defined physical point, specified by latitude and longitude, that has been determined by a survey, conducted in accordance with the guidance provided in this manual. Communication facilities, gates, navaids, navigation checkpoints, obstacles and runway thresholds are usually surveyed points.

b) *Calculated point.* A calculated point is a point in space that need not be specified explicitly in latitude and longitude, but that has been derived, by mathematical manipulation, from a known surveyed point. A fix, specified by radial/bearing and range from a known surveyed point such as a navaid or by the intersection of a number of radial/bearings from a number of navaids, is an example of a calculated point. En-route way-points, which are computed from the intersection of routes or from cross radial fixes on routes, are also calculated points, albeit they are reported in latitude and longitude.

c) *Declared point.* A declared point is a point in space, defined by latitude and longitude, that is not dependent upon, nor formally related to, any known surveyed point. Flight information region (FIR) boundary points and prohibited, restricted or danger area points that are outside control areas are often declared points.

2.3 SOURCE OF RAW AERONAUTICAL DATA

It is the responsibility of relevant technical services, within the appropriate authority of a Contracting State, to ensure the determination of raw aeronautical data required for promulgation by the aeronautical information service (AIS). On receipt of the raw data, the relevant technical services must check, record and edit the data so that they can be released to the next intended user in a standard format. Raw aeronautical data containing positional information can originate from a number of different sources.

a) *En-route.* The surveyed positions of navaids and communication facilities are normally provided by the owner/operator (ATC) of the equipment.

b) *SID, STAR, Instrument approach procedures.* The calculated positions are normally determined by the air traffic service provider responsible for the procedure, in coordination with the technical branch dealing with the procedure design within the State aviation authority.

c) *Aerodrome/heliport.* The surveyed positions of thresholds, gates, obstacles and navaids, etc. located at the aerodrome/heliport are normally provided by the owner or operator of the aerodrome/heliport.

d) *Airspace divisions and restrictions*. The declared positions are normally defined by State civil aviation or military authorities or other government bodies.

2.4 ACCURACY REQUIREMENTS

2.4.1 For aeronautical data to be usable, it must be accurate and, in this context, can be subdivided into two distinct categories:

a) evaluated aeronautical data; and

b) reference aeronautical data.

2.4.2 Evaluated aeronautical data include such information as positional data, elevation, runway length, declared distances, platform-bearing characteristics and magnetic variation. Reference aeronautical data include navaid identifiers, navaid frequencies, way-point names, rescue and fire-fighting facilities, hours of operation, telephone numbers, etc.

2.4.3 The accuracy requirement for the reference data is absolute; the information is either correct or incorrect. Conversely, the required degree of accuracy of the evaluated data will vary depending upon the use to which the data are put. This manual addresses primarily evaluated positional data but many of the procedures may be applied to other evaluated aeronautical data and to reference data, if required. Tables 2-1 to 2-5 contain accuracy requirements for aeronautical data as specified in Annex 11 and in Annex 14, Volumes I and II. The requirements for quality assurance and aeronautical data processing procedures are provided in more detail in Chapter 6.

Definition of ***Accuracy.*** A degree of conformance between the estimated or measured value and the true value.

Note.— For measured positional data the accuracy is normally expressed in terms of a distance from a stated position within which there is a defined confidence of the true position falling.

2.4.4 Accuracy requirements are based upon a 95% confidence level (see Table 2-6). The underlying statistical distribution for positional data in two dimensions is usually taken to be the circular normal distribution. The probability P of a point actually falling within a circle of radius $c\sigma$ around its reported position, where σ represents the standard univariate deviation and c is a numeric coefficient, is:

$$P = 1 - \exp(-c^2/2).$$

2.4.5 The Circular Error Probable (CEP) is the radius of the circle within which 50% of the measurements lie, that is, 1.1774 σ. The radius within which 95% of the measurements lie is 2.448 δ or 2.079 × CEP. Table 2-6 relates σ error values, probable errors and probabilities in one, two and three dimensions.

2.4.6 The RNP types (see Table 2-7) specify the navigation performance accuracy of all the user and navigation system combinations within an airspace. RNP types can be used by airspace planners to determine airspace utilization potential and as input for defining route widths and traffic separation requirements, although RNP by itself is not a sufficient basis for setting a separation standard.

2.5 RESOLUTION REQUIREMENTS

Definition of ***Resolution.*** A number of units or digits to which a measured or calculated value is expressed and used.

2.5.1 Resolution of positional data is the smallest separation that can be represented by the method employed to make the positional statement. Care must be taken that the resolution does not affect accuracy; the resolution is always a rounded value as opposed to a truncated value. The order of publication and the charting resolution of aeronautical data must be that specified in Tables 2-1 to 2-5.

Definition of ***Precision.*** The smallest difference that can be reliably distinguished by a measurement process.

Note.— In reference to the geodetic surveys, precision is a degree of refinement in performance of an operation or a degree of perfection in the instruments and methods used when making measurements.

2.5.2 The terms "precision" and "resolution" are often interchangeable in general use. Here precision is a measure of the data field capacities that are available within a specific system design. (Example: 54° 33' 15" is expressed to a resolution of one second.) Any process that manipulates data subsequent to the original measurement or definition cannot increase the precision to which the data were originally measured or defined, regardless of the resolution available within the system itself.

2.6 INTEGRITY REQUIREMENTS

Definition of ***Integrity (aeronautical data)***. A degree of assurance that an aeronautical data and its value has not been lost nor altered since the data origination or authorized amendment.

2.6.1 General

2.6.1.1 The integrity of the data can be regarded as the degree of assurance that any data item retrieved from a storage system has not been corrupted or altered in any way since the original data entry or its latest authorized amendment. This integrity must be maintained throughout the data process from survey to data application. In respect to AIS, integrity must be maintained to the next intended user.

2.6.1.2 Integrity is expressed in terms of the probability that a data item, retrieved from a storage system with no evidence of corruption, does not hold the same value as intended. For example, an integrity of 1×10^{-8} means that an undetected corruption can be expected in no more than one data item in every 100 000 000 data items processed. Loss of integrity does not necessarily mean loss of accuracy. However, it does mean that it is no longer possible to prove that the data are accurate without a further verification of the data from the point at which integrity can be confirmed.

2.6.1.3 The integrity requirements for data are not absolute. The risk associated with a point being in error is dependent upon how that data point is being used. Thus, the integrity of a point at threshold used for landing needs a higher integrity than one used for en-route guidance. It is important to note that a lower accuracy does not necessarily imply a lower integrity requirement.

2.6.2 Requirement for integrity

2.6.2.1 A data item's use forms the basis for determining its integrity requirement. Aeronautical data integrity requirements must therefore be based upon the potential risk resulting from the corruption of data and upon the particular use of the data item. Consequently, the following classification of data integrity must apply.

a) *Critical data.* There is a high probability when using corrupted critical data that the continued safe flight and landing of an aircraft would be severely at risk with the potential for catastrophe.

b) *Essential data.* There is a low probability when using corrupted essential data that the continued safe flight and landing of an aircraft would be severely at risk with the potential for catastrophe.

c) *Routine data.* There is a very low probability when using corrupted routine data that the continued safe flight and landing of an aircraft would be severely at risk with the potential for catastrophe.

2.6.2.2 To each of these types of data, an integrity level requirement has been assigned as follows.

a) *Critical data:* 1×10^{-8}. This level is given to the runway threshold data which define the landing point. The level of integrity has been derived from the integrity requirement for autoland and is defined to ensure that the overall process, of which aeronautical data are only a part, has the required integrity.

b) *Essential data:* 1×10^{-5}. This level is assigned to points which, while an error can in itself result in an aircraft being outside of the envelope required, excursion does not necessarily result in a catastrophe. Examples include en-route navigation aids and obstacles. The reason why obstacle data can be held with a relatively lower level of integrity is that, while the data need to be accurate at the time the procedures are designed, any subsequent corruption should have no impact on the safety of the aircraft on the condition that it conforms to the procedure requirements.

c) *Routine data:* 1×10^{-3}. This level is assigned to data for which errors do not affect the navigation performance. These include FIR boundary points.

Note.— A classification of aeronautical data with respect to integrity are provided in Tables 2-1 to 2-5.

TABLES FOR CHAPTER 2

Table 2-1. Aeronautical data quality requirements (latitude and longitude)

Latitude and longitude	*Accuracy data type*	*Publication resolution*	*Chart resolution*	*Integrity classification*
Flight information region boundary points	2 km (1 NM) declared	1 min	as plotted	1×10^{-3} routine
P, R, D area boundary points (outside CTA/CTZ boundaries)	2 km (1 NM) declared	1 min	as plotted	1×10^{-3} routine
P, R, D area boundary points (inside CTA/CTZ boundary)	100 m calculated	1 sec	as plotted	1×10^{-5} essential
CTA/CTZ boundary points	100 m calculated	1 sec	as plotted	1×10^{-5} essential
En-route NAVAIDS and fixes, holding, STAR/SID points	100 m surveyed/ calculated	1 sec	1 sec	1×10^{-5} essential
Obstacles en-route	100 m surveyed	1 sec	as plotted	1×10^{-3} routine
Aerodrome/heliport reference point	30 m surveyed/ calculated	1 sec	1 sec	1×10^{-3} routine
NAVAIDS located at the aerodrome/ heliport	3 m surveyed	1/10 sec	as plotted	1×10^{-5} essential
Obstacles in the circling area and at the aerodrome/heliport	3 m surveyed	1/10 sec	1/10 sec (AOC Type C)	1×10^{-5} essential
Significant obstacles in the approach and take-off area	3 m surveyed	1/10 sec	1/10 sec (AOC Type C)	1×10^{-5} essential
Final approach fixes/points and other essential fixes/points comprising instrument approach procedures	3 m surveyed/ calculated	1/10 sec	1 sec	1×10^{-5} essential
Runway threshold	1 m surveyed	1/100 sec	1 sec	1×10^{-8} critical
Runway end (flight path alignment point)	1 m surveyed	1/100 sec	—	1×10^{-8} critical
Runway centre line points	1 m surveyed	1/100 sec	1/100 sec	1×10^{-8} critical
Taxiway centre line points	0.5 m surveyed	1/100 sec	1/100 sec	1×10^{-5} essential
Ground taxiway centre line points, air taxiways and transit routes points	0.5 m surveyed/ calculated	1/100 sec	1/100 sec	1×10^{-5} essential
Aircraft/helicopter standpoints/INS checkpoints	0.5 m surveyed	1/100 sec	1/100 sec	1×10^{-3} routine
Geometric centre of TLOF or FATO thresholds, heliports	1 m surveyed	1/100 sec	1 sec	1×10^{-8} critical

Table 2-2. Aeronautical data quality requirements (elevation/altitude/height)

Elevation/altitude/height	*Accuracy data type*	*Publication resolution*	*Chart resolution*	*Integrity classification*
Aerodrome/heliport elevation	0.5 m or 1 ft surveyed	1 m or 1 ft	1 m or 1 ft	1×10^{-5} essential
WGS-84 geoid undulation at aerodrome/heliport elevation position	0.5 m or 1 ft surveyed	1 m or 1 ft	1 m or 1 ft	1×10^{-5} essential
Runway or FATO threshold, non-precision approaches	0.5 m or 1 ft surveyed	1 m or 1 ft	1 m or 1 ft	1×10^{-5} essential
WGS-84 geoid undulation at runway or FATO threshold, TLOF geometric centre, non-precision approaches	0.5 m or 1 ft surveyed	1 m or 1 ft	1 m or 1 ft	1×10^{-5} essential
Runway or FATO threshold, precision approaches	0.25 m or 1 ft surveyed	0.5 m or 1 ft	0.5 m or 1 ft	1×10^{-8} critical
WGS-84 geoid undulation at runway or FATO threshold, TLOF geometric centre, precision approaches	0.25 m or 1 ft surveyed	0.5 m or 1 ft	0.5 m or 1 ft	1×10^{-8} critical
Obstacle Clearance Altitude/Height (OCA/H)	as specified in PANS-OPS (Doc 8168)	—	as specified in PANS-OPS (Doc 8168)	1×10^{-5} essential
Threshold crossing height, precision approaches	0.5 m or 1 ft calculated	0.5 m or 1 ft	0.5 m or 1 ft	1×10^{-8} critical
Obstacles in the approach and take-off areas	1 m or 1 ft surveyed	1 m or 1 ft	1 m or 1 ft	1×10^{-5} essential
Obstacles in the circling areas and at the aerodrome/heliport	1 m or 1 ft surveyed	1 m or 1 ft	1 m or 1 ft	1×10^{-5} essential
Obstacles en-route, elevations	3 m (10 ft) surveyed	3 m (10 ft)	3 m (10 ft)	1×10^{-3} routine
Distance Measuring Equipment/ Precision (DME/P)	3 m (10 ft) surveyed	3 m (10 ft)	—	1×10^{-5} essential
Distance Measuring Equipment (DME) elevation	30 m (100 ft) surveyed	30 m (100 ft)	30 m (100 ft)	1×10^{-5} essential
Instrument approach procedures altitude	as specified in PANS-OPS Doc 8168)	—	as specified in PANS-OPS (Doc 8168)	1×10^{-5} essential
Minimum altitudes	50 m or 100 ft calculated	50 m or 100 ft	50 m or 100 ft	1×10^{-3} routine

Table 2-3. Aeronautical data quality requirements (declination and magnetic variation)

Declination/variation	*Accuracy data type*	*Publication resolution*	*Chart resolution*	*Integrity classification*
VHF NAVAID station declination used for technical line-up	1 degree surveyed	1 degree	—	1×10^{-5} essential
NDB NAVAID magnetic variation	1 degree surveyed	1 degree	—	1×10^{-3} routine
Aerodrome/heliport magnetic variation	1 degree surveyed	1 degree	1 degree	1×10^{-5} essential
ILS localizer antenna magnetic variation	1 degree surveyed	1 degree	—	1×10^{-5} essential
MLS azimuth antenna magnetic variation	1 degree surveyed	1 degree	—	1×10^{-5} essential

Table 2-4. Aeronautical data quality requirements (bearing)

Bearing	*Accuracy data type*	*Publication resolution*	*Chart resolution*	*Integrity classification*
Airway segments	1/10 degree calculated	1 degree	1 degree	1×10^{-3} routine
En-route and terminal fix formations	1/10 degree calculated	1/10 degree	1/10 degree	1×10^{-3} routine
Terminal arrival/departure route segments	1/10 degree calculated	1 degree	1 degree	1×10^{-3} routine
Instrument approach procedure fix formations	1/100 degree calculated	1/100 degree	1/10 degree	1×10^{-5} essential
ILS localizer alignment	1/100 degree surveyed	1/100 degree True	1 degree	1×10^{-5} essential
MLS zero azimuth alignment	1/100 degree surveyed	1/100 degree True	1 degree	1×10^{-5} essential
Runway and FATO bearing	1/100 degree surveyed	1/100 degree True	1 degree	1×10^{-3} routine

Table 2-5. Aeronautical data quality requirements (length/distance/dimension)

Length/distance/dimension	*Accuracy data type*	*Publication resolution*	*Chart resolution*	*Integrity classification*
Airway segments length	1/10 km or 1/10 NM calculated	1/10 km or 1/10 NM	1 km or 1 NM	1×10^{-3} routine
En-route fix formations distance	1/10 km or 1/10 NM calculated	1/10 km or 1/10 NM	2/10 km (1/10 NM)	1×10^{-3} routine
Terminal arrival/departure route segments length	1/100 km or 1/100 NM calculated	1/100 km or 1/100 NM	1 km or 1 NM	1×10^{-5} essential
Terminal and instrument approach procedure fix formations distance	1/100 km or 1/100 NM calculated	1/100 km or 1/100 NM	2/10 km (1/10 NM)	1×10^{-5} essential
Runway and FATO length, TLOF dimensions	1 m or 1 ft surveyed	1 m or 1 ft	1 m (AD chart) 0.5 m (AOC chart)	1×10^{-8} critical
Stopway length	1 m or 1 ft surveyed	1 m or 1 ft	0.5 m (AOC chart)	1×10^{-8} critical
Landing distance available	1 m or 1 ft surveyed	1 m or 1 ft	1 m (AD chart) 0.5 m (AOC chart)	1×10^{-8} critical
ILS localizer antenna — runway end and FATO end, distance	3 m or 10 ft calculated	3 m (10 ft)	as plotted	1×10^{-3} routine
ILS glide slope antenna — threshold, distance along centre line	3 m or 10 ft calculated	3 m (10 ft)	as plotted	1×10^{-3} routine
ILS markers — threshold distance	3 m or 10 ft calculated	3 m (10 ft)	2/10 km (1/10 NM)	1×10^{-5} essential
ILS DME antenna — threshold, distance along centre line	3 m or 10 ft calculated	3 m (10 ft)	as plotted	1×10^{-5} essential
MLS azimuth antenna — runway end and FATO end, distance	3 m or 10 ft calculated	3 m (10 ft)	as plotted	1×10^{-3} routine
MLS elevation antenna — threshold, distance along centre line	3 m or 10 ft calculated	3 m (10 ft)	as plotted	1×10^{-3} routine
MLS DME/P antenna — threshold, distance along centre line	3 m or 10 ft calculated	3 m (10 ft)	as plotted	1×10^{-5} essential

Table 2-6. Accuracy and probability

Accuracy expression	*One-dimensional probability*	*Two-dimensional probability*	*Three-dimensional probability*
Three sigma	99.7%	98.9%	97.1%
Two sigma	95.0%	86.0%	78.8%
One sigma	68.0%	39.3%	19.9%
Probable error	50.0% (0.67 σ)	50.0% (1.18 σ)	50.0% (1.54 σ)

Table 2-7. RNP types

Accuracy	*RNP 1*	*RNP 4*	*RNP 12.6*	*RNP 20*
95% position accuracy in the designated airspace	±1.85 km (±1.0 NM)	±7.4 km (±4.0 NM)	±23.3 km (±12.6 NM)	±37 km (±20.0 NM)

Chapter 3

THE GLOBAL WGS-84 COORDINATE SYSTEM

3.1 DEFINITION OF THE WGS-84 COORDINATE SYSTEM

3.1.1 The World Geodetic System — 1984 (WGS-84) coordinate system is a Conventional Terrestrial System (CTS), realized by modifying the Navy Navigation Satellite System (NNSS), or TRANSIT, Doppler Reference Frame (NSWC 9Z-2), in origin and scale, and rotating it to bring its reference meridian into coincidence with the Bureau International de l'Heure (BIH)-defined zero meridian.

3.1.2 Origin and axes of the WGS-84 coordinate system are defined as follows.

a) *Origin* is the earth's centre of mass.

b) *Z-axis* is the direction of the Conventional Terrestrial Pole (CTP) for polar motion, as defined by BIH on the basis of the coordinates adopted for the BIH stations.

c) *X-axis* is the intersection of the WGS-84 reference meridian plane and the plane of the CTP's equator, the reference meridian being the zero meridian defined by the BIH on the basis of the coordinates adopted for the BIH stations.

d) *Y-axis* completes a right-handed, earth-centred, earth-fixed (ECEF) orthogonal coordinate system, measured in the plane of the CTP equator, 90° east of the X-axis.

Note.— An illustration of the WGS-84 coordinate system origin and axes, which serve also as the geometric centre and the X, Y, and Z axes of the WGS-84 ellipsoid, is given in Figure 3-1.

3.1.3 WGS-84 is an earth-fixed global reference frame, including an earth model, and is defined by a set of primary and secondary parameters. The primary parameters, given in Table 3-1, define the shape of an earth ellipsoid, its angular velocity, and the earth mass which is included in the ellipsoid of reference.

3.1.4 The secondary parameters define a detailed Earth Gravity Field Model (EGFM) of the degree and order $n = m = 180$. The WGS-84 EGFM, through $n = m = 180$, is to be used when calculating WGS-84 geoid heights, WGS-84 gravity disturbance components, and WGS-84 1° × 1° mean gravity anomalies via spherical harmonic expansions. Expansions to this degree and order ($n = m = 180$) are needed to accurately model variations in the earth's gravitational field on or near the earth's surface. The WGS-84 EGFM, through $n = m = 41$, is more appropriate for satellite orbit calculation (e.g. GPS navigation satellites) and prediction purposes.

3.2 REALIZATION OF THE WGS-84 COORDINATE SYSTEM

3.2.1 The origin and the orientation of coordinate axes in WGS-84 are defined by the X, Y, Z coordinates of the five GPS monitoring stations (see Figure 3-2).

3.2.2 Historically, the coordinates of the GPS tracking sites have been determined by the use of Doppler measurements to the TRANSIT satellite navigation system. Data, observed over long periods, have been processed in order to derive precise station coordinates. The use of TRANSIT Doppler measurements in WGS-84 is a good example of the practical realization of a reference system. It should be pointed out, however, that errors can spread into the procedures used to realize reference frames.

3.3 ACCURACY OF WGS-84 COORDINATES

3.3.1 The accuracy, one sigma, (1 σ) of WGS-84 coordinates directly determined in WGS-84 by GPS Satellite Point Positioning, their respective precise ephemerides and ground-based satellite tracking data acquired in static mode, in terms of geodetic latitude ϕ, geodetic longitude λ, and geodetic height h, is:

a) horizontal — $\sigma\phi = \sigma\lambda = \pm 1$ m (1 σ); and

b) vertical — $\sigma h = \pm 1 \ldots 2$ m (1 σ).

3.3.2 These errors incorporate not only the observational error but also the errors associated with placing the origin of the WGS-84 coordinate system at the earth's centre of mass and with determining the correct scale. These absolute values should not be confused with the centimetre-precision of GPS differential positioning. At the time WGS-84 was established, only satellite Doppler measurements with corresponding accuracy were available to determine the ground control segment of WGS-84.

3.3.3 The WGS-84 coordinates of a non-satellite-derived local geodetic network station will be less accurate than the WGS-84 coordinates of a GPS station. This is due to the distortions and surveying errors present in local geodetic datum networks, i.e. the lack of a sufficient number of properly placed GPS stations collocated with local geodetic networks for use in determining the transformation parameters and the uncertainty introduced by the datum transformation.

3.3.4 The accuracy of ±1 m in the definition of WGS-84 is sufficient for nearly all air navigation applications. Additional considerations may be necessary if, for example, satellite-based landing systems down to Category III are to be used in the future. Precision approach Category III needs a vertical accuracy of 0.6 m and a horizontal accuracy of 6.0 m, which cannot be supplied by WGS-84 according to its accuracy definition, but can be supplied, for instance, by International Terrestrial Reference Frame (ITRF).

Table 3-1. Parameters of WGS-84

Parameter	*Abbreviation*	*WGS-84*
Semi-major axis	a	6 378 137 m
Angular velocity	ω	7.292115×10^{-5} rad s^{-1}
Geocentric gravitational constant (Mass of the earth's atmosphere included)	GM	398 600.5 km^3 s^{-2}
Normalized second degree zonal harmonic coefficient of the gravitational potential	$\overline{C}_{2,0}$	$-484.16685 \times 10^{-6}$
Flattening (derived from $\overline{C}_{2,0}$)	f	1/298.257223563

FIGURES FOR CHAPTER 3

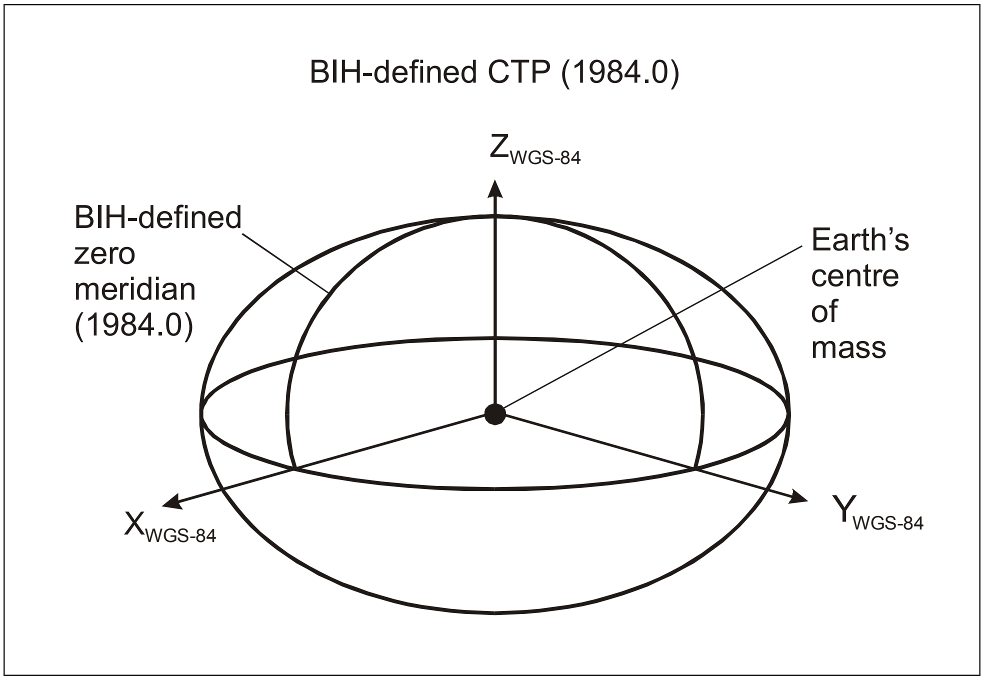

Figure 3-1. The WGS-84 coordinate system definition

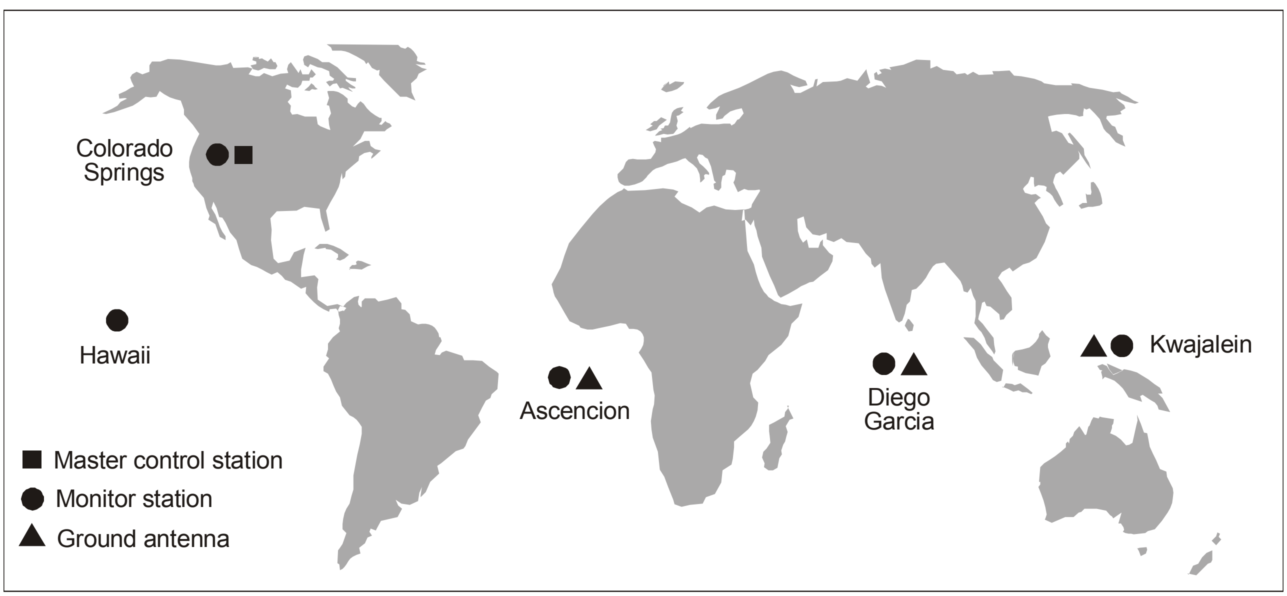

Figure 3-2. Realization of origin and orientation of WGS-84

Chapter 4

A GUIDE TO OBTAIN WGS-84 COORDINATES

4.1 GENERAL

This chapter is intended to be a guide with "recipes" for users to produce WGS-84 coordinates. The user of the manual is guided through the steps necessary, depending on the existence and the quality of aeronautical data, to obtain WGS-84 coordinates. The first question to be answered is:

Are coordinates of the required accuracy available?

Yes If these coordinates are available in a local reference frame, then proceed to 4.2 (Scenario 1). If the available coordinates have been digitized from maps, then proceed to 4.4 (Scenario 3).

When considering the use of existing data, it is important to check and control the coordinates of navigation facilities with respect to accuracy and integrity before transforming them to WGS-84 by mathematical means. One has to keep in mind that coordinates in air navigation could be safety-critical and that high-integrity requirements have to be fulfilled. In order to fulfil the quality requirements for coordinates, the surveyor must ensure that:

a) point labels have not been interchanged or misidentified;

b) the coordinates can be verified by aid of redundant measurements; and

c) the accuracy is predictable and sufficient.

No If coordinates of the required accuracy are not available or if, for example, the integrity requirements cannot be fulfilled, a resurvey with related field work must be performed. The different methods of performing this resurvey to provide accurate WGS-84 coordinates are explained under 4.3 (Scenario 2).

4.2 SCENARIO 1: COORDINATES IN A LOCAL REFERENCE FRAME ARE AVAILABLE

Two approaches exist to transform coordinates given in national coordinates to WGS-84 coordinates. They are dependent on knowledge of the transformation parameters and the type of coordinate system, and can be used as stand-alone methods or combined.

4.2.1 Checking the type of coordinate system

4.2.1.1 Before carrying out a datum transformation, check if the transformation parameters from the local reference frame to WGS-84 are known and answer the following question:

Are all transformation parameters known?

Yes Perform a computational datum transformation by using the datum transformation formulae (4.2.2 refers) to determine the WGS-84 coordinates. Several software programmes exist to support this procedure, e.g. the DATUM programme.

Note.— DATUM performs coordinate transformations between a variety of existing geodetic reference frames and WGS-84.

No Use the GPS surveying technique, at known control stations (covering the area under consideration), to obtain WGS-84 coordinates. Since these control stations are known in the local reference frame and in WGS-84, two sets of coordinates for identical stations exist. These can then be used to determine the datum parameters in the Helmert Formula. At least three known control stations have to be surveyed by GPS to obtain additional WGS-84 coordinates necessary for determining all seven Helmert transformation parameters (using the Inverse Helmert Formula). In practice, however, it is usual to have as many common points as possible to obtain the best estimate of the parameters by least squares.

Note.— See Appendix A for information on GPS surveying. See Appendix D for a detailed description of the Helmert Formula.

4.2.1.2 For the following example, it is assumed that only the shifts of origin between the local reference frame and the WGS-84 have to be determined and that, therefore, only one known control station was GPS-surveyed. The inverse Helmert Formula for solving the three shift of origin parameters reads:

$$\underset{\text{Shift of origin}}{\begin{bmatrix} \Delta X \\ \Delta Y \\ \Delta Z \end{bmatrix}} = \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{WGS\text{-}84} - \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{Local}$$

Assumption: No change in orientation ($\varepsilon_X = \varepsilon_Y = \varepsilon_Z = 0$) and scale ($\mu = 1$) between the local reference frame and WGS-84.

4.2.1.3 After determining all necessary transformation parameters, proceed as explained at the beginning of 4.2.1.1. The method of referencing a local (i.e. relative) and sufficiently accurate GPS aerodrome survey to WGS-84 by simply measuring the coordinate differences between one aerodrome point to a known and monumented WGS-84 station is called direct geodetic connection. In applying this procedure, all the airport coordinates can be directly transformed to WGS-84. The problem, however, is that not many geodetic stations exist worldwide for which accurate WGS-84 coordinates are known. It is therefore recommended to use, if available, ITRF stations for this purpose. If no ITRF station is near the aerodrome or navigation facility, the connection survey can be very laborious. In this case, long distances have to be traversed by surveying, which could be very expensive.

4.2.2 Horizontal datum transformations

4.2.2.1 There are three different approaches for transforming coordinates from a local datum to WGS-84. Use the:

a) *Helmert Formula* to carry out the transformation in rectangular Cartesian coordinates X, Y, Z, using three-, four-, or seven-parameter transformations depending on the availability and/or reliability of the transformation parameters. The Helmert Formula can also be applied for spatial ellipsoidal coordinates f, l, *h* by transforming from ellipsoidal coordinates to rectangular coordinates and vice versa.

b) *Standard Molodensky Formula* to solve the transformation in curvilinear coordinates f, l, *h*.

c) *Multiple Regression Equation* approach to account for the non-linear distortion in the local geodetic datum. (Only recommended for large areas).

Note.— See Appendix D for a detailed description of these datum transformation formulae and Table B-1 for a list of reference ellipsoids and parameters.

4.2.2.2 *Accuracy considerations*

4.2.2.2.1 Because of error propagation, a datum transformation will never improve the survey accuracy. In most cases the accuracy of the transformed coordinates in the absolute sense is worse than the accuracy of the original coordinates. The user has to check, in particular, whether the resulting coordinate accuracy still meets the requirements. Furthermore, this quality control may be difficult to perform.

4.2.2.2.2 Two of the several reasons known for this loss of accuracy are:

a) because the datum or transformation parameters are, in many cases, only weakly determined, substantial discrepancies of up to 50 m in datum parameters can occur between published values from different reference sources. Furthermore, the accuracy of a navigation aid's original coordinates may not be sufficient, and in many cases the accuracy of the datum parameters is undefined; and

b) there may be a slight distortion in the national network in the area under consideration.

4.2.2.3 *Limitations of transformations*

4.2.2.3.1 It should be noted that random and systematic errors in local survey data transform directly to WGS-84. Because of geodetic network geometry and error propagation in these networks, the local datum parameters for a State are not constant in practice but vary with location in the geodetic network.

4.2.2.3.2 The signal-to-noise ratio for the datum parameters is, in many cases, close to one, i.e. the noise level is very high relative to the magnitude of the datum parameter itself. For example, the orientation angles of a datum could be published, typically as, say, 0.5" ± 0.3".

4.2.2.3.3 The predicted error, or uncertainty, is often larger than the value itself. Figure 4-1 shows, in a qualitative manner, how the errors in transformation procedures propagate into transformed coordinates.

4.2.2.3.4 The error sources in a datum transformation are errors in the shift parameters, the orientation parameters and the scale factor. The scale factor error is incorporated in the above in that it is treated like an orientation error, in radians, applied to the coordinate value to be transformed by multiplication. Even an accurate survey with an internal accuracy of, say, 0.1 m may show, after the transformation parameters have been applied, only 1 m level accuracy in WGS-84. Here the difference between absolute and relative point accuracy has to be considered.

4.3 SCENARIO 2: COORDINATES OF THE REQUIRED ACCURACY ARE NOT AVAILABLE

4.3.1 New field survey techniques

4.3.1.1 If coordinates of the required accuracy are not available, a new field survey must be performed using one, or a combination of, the following techniques:

a) conventional surveying;

b) GPS surveying; or

c) photogrammetric surveying.

4.3.2 Best technique(s) for new field survey

4.3.2.1 In deciding which of the above techniques is (or are) the most efficient for the new field survey, the following may be used as guidance.

a) Use the GPS technique for surveying limited and relatively small areas in a very economical way.

b) Use the photogrammetric technique if the area to be surveyed is very large.

c) Use conventional surveying if the area to be surveyed contains many obstructions which would lead to GPS signal losses or multipath.

Note.— There is no doubt that a complete resurvey of the point using differential GPS satellite surveying techniques (relative to a known station with WGS-84 coordinates) is the most accurate approach for determining precise WGS-84 coordinates.

4.3.3 Determination by conventional terrestrial surveying

4.3.3.1 Figure 4-2 shows how WGS-84 coordinates can be obtained by conventional terrestrial surveying.

4.3.3.2 Some of the conventional surveying instruments of modern type (levelling instrument, theodolite, distance meter, total station) have interactive field computational capabilities. After downloading the data via an interface into an office computer, final post-processing could be carried out. Before the derived coordinates can enter the survey database, they have to be quality-controlled, and integrity checks have to be performed. Various graphic visualizations of data and results can also be done.

Note.— For more information on conventional surveying, see Appendix E.

4.3.4 Determination by GPS surveying

4.3.4.1 Most of the field surveying which is necessary for the positioning of navigation aids, radars, runways, etc., is best carried out by differential GPS satellite surveying. This method has the advantages of 24-hour all-weather operations, ease of use, speed, economy, high accuracy and, most importantly, direct compatibility with the WGS-84 datum.

4.3.4.2 GPS receivers store the field data. After finishing the survey, the data have to be downloaded to a computer where they are post-processed using software packages provided by GPS hardware manufacturers and/or universities. The processing can be done either by individual baseline or in a multisession-multistation network approach. Again, before the derived coordinates can enter the survey database, they have to be quality-controlled, and integrity checks have to be performed. Various visualizations of data and results can also be performed.

Note.— For the choice of the GPS surveying technique, (static, rapid static, kinematics survey, etc.) which depends to a great extent on the desired accuracy, see Appendix A and Table A-2.

4.3.5 Determination by aero-photogrammetry

4.3.5.1 Figure 4-3 shows the determination of WGS-84 coordinates by using photogrammetric flights. The parameters of the photogrammetric flight have to be determined as a function of anticipated coordinate accuracy of the ground stations. If no WGS-84 coordinates at ground stations are available, they have to be established using GPS differential surveying techniques. Thus, a network of ground control points whose coordinates and heights are known in advance is an essential requirement for referencing the newly derived coordinates to a national datum. The points to be coordinated have to be marked so that a unique identification in the aerial photos is possible.

4.3.5.2 If necessary, permission must be obtained to release photo data. These data are then developed and the stereo model construction is carried out in a photogrammetric instrument in the office. After inputting the ground control coordinates and, if available, GPS-derived camera positions, the data are processed by a bundle block adjustment.

4.3.5.3 Again, before the derived coordinates can enter the survey database, they have to be quality-controlled, and integrity checks have to be performed. Various visualizations of data and results can then be done.

Note.— For more information on aero-photogrammetry and the minimization of ground control stations, see Appendix E.

4.4 SCENARIO 3: COORDINATES DIGITIZED FROM MAPS ARE AVAILABLE

Note.— This section helps the user transform coordinates to WGS-84, if the coordinates are available from digitized maps.

4.4.1 Restrictions

4.4.1.1 While digitized data have no inherent scale information, the accuracy of the data is clearly limited by the corresponding accuracy of the analogue map from which the data were originally extracted and of the digitizing process involved. A new analogue map may be printed using a larger scale than that of the original map, but this does not increase the accuracy to the level normally associated with the larger scale. The problem is further compounded by the frequent revision and updating of the database with newly surveyed field data. Furthermore, the digitizing process involves the straightening and squaring of regular objects, leading to apparently "well drawn" maps even after the enlargement process.

4.4.1.2 The most important drawback of digitized maps, however, is the very nature of an analogue database. High-precision mapping coordinates are generally given in national grid northings and eastings, which have been obtained by converting geodetic (ellipsoidal) coordinates into a map projection. In addition, one must consider the more significant projection scale and orientation errors inherent in all map projections. While these can be reduced by the judicious use of an orthomorphic projection (e.g. Transverse Mercator), they are still substantial, rendering the process of extracting coordinate information from a map precarious. For example, if the grid coordinates of two points are extracted from the map and the grid distance computed, the distance would be up to 30 cm per kilometre shorter than the value measured on the ground. This is significant and may have serious implications.

4.4.1.3 Therefore, when digitizing coordinates from maps, the following points should be considered:

a) It is necessary to check by which technique the map was established (from analogue data/digitizing of other maps, from digital data, etc.).

b) In order to convert the northings and eastings to geographical coordinates, it is necessary to know the exact formulae of the map projection.

c) It is necessary to know the original datum of the projected coordinates as well as the new one, when transforming.

d) Datum coordinate transformation can only be applied after converting map projection coordinates to geodetic coordinates.

e) The resulting accuracy of digitized coordinates should be checked and verified in order to decide whether the anticipated accuracy requirements have been met.

4.4.1.4 WGS-84 is the definition of the centre of mass of the Earth as determined in 1984 and all charts produced prior to that date using a different geodetic reference will not correspond exactly to new charts based on WGS-84. Finally, one has to bear in mind that maps never contain ellipsoidal heights. For example, heights in different maps may refer to:

a) different zero points (tide gauges); or

b) different types of height systems. (There are not only orthometric heights, but also so-called normal heights, for example, in Eastern Europe.)

4.4.2 Transformations

4.4.2.1 To transform coordinates digitized from maps to WGS-84, it is essential to answer the following question:

Is the kind of map projection, which was used in the local survey for mapping the reference ellipsoid to the plane and for computing the plane metric coordinates, known?

No If this question cannot be answered or the projection is not known, then Scenario 2 applies and a resurvey should be performed.

Yes If the type of map projection is known, the inverse map projection has to be calculated to compute latitude and longitude of the digitized metric coordinates on the reference ellipsoid.

Note.— See Appendix F for different types of map projections.

4.4.2.2 All datum transformations require the use of the ellipsoidal height h in the local system, which is:

$$h = H + N$$

where H is the orthometric height (elevation) and N is the geoid undulation (height). In general, only the orthometric height is known (and found also on maps). The geoid undulation has to be taken from a digital model (if available). However, an investigation was made checking the effect of an unknown (orthometric) height on the transformed latitude and longitude of a point using the Helmert Formulae. By assigning heights ranging from 0 m to 8 000 m, it was concluded that the effect on both latitude and longitude was negligible (less than 15 cm at 8 000 m). Consequently, for a point of known latitude and longitude but unknown (orthometric) height, an arbitrary height of 0 m could be assigned without significantly affecting the transformation.

4.4.2.3 Because national surveying agencies are using different kinds of reference ellipsoids, the next step is to determine which reference ellipsoid is being used in order to perform the datum transformation from the local datum to the global datum. Sometimes it may be possible to transform directly from the local datum to WGS-84. If not, then a further transformation from the global datum to WGS-84 has to be undertaken.

FIGURES FOR CHAPTER 4

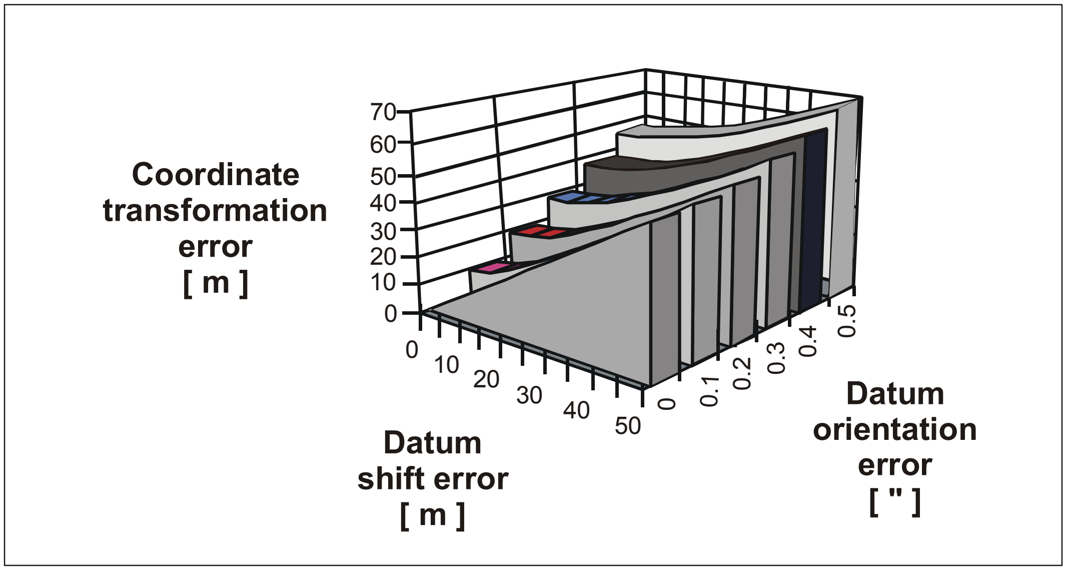

Figure 4-1. Error propagation in datum transformation

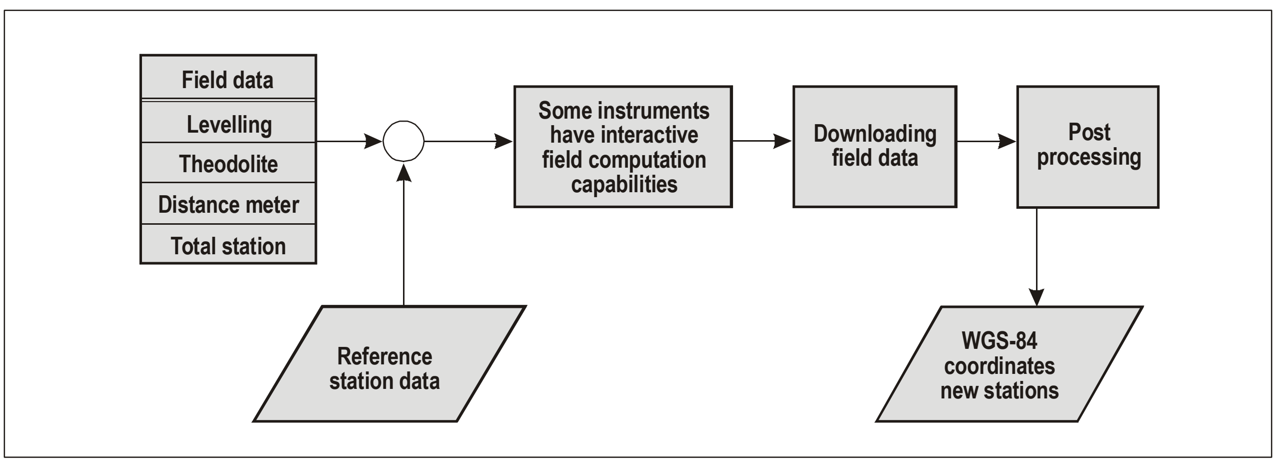

Figure 4-2. From terrestrial surveying data to WGS-84 coordinates

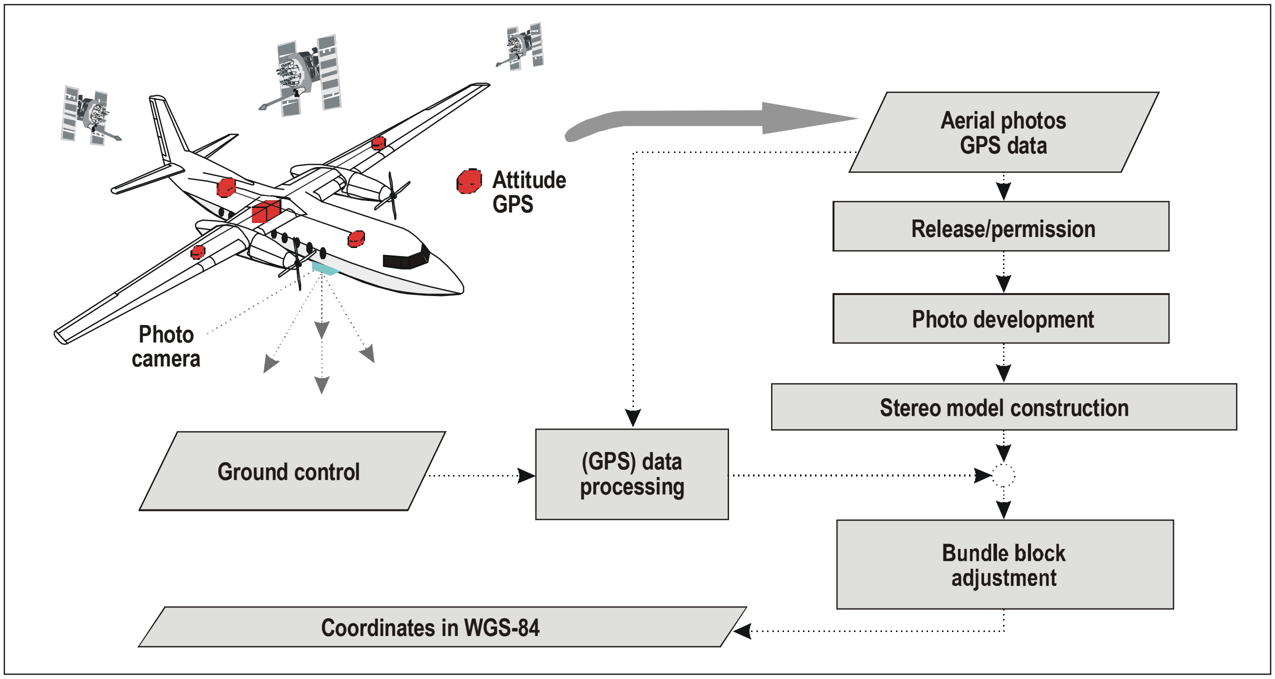

Figure 4-3. From results of photogrammetric flights to WGS-84 coordinates

Chapter 5

SURVEYING GUIDANCE

5.1 INTRODUCTION

This chapter sets out guidance for surveying the geographical positions of navigation aids and navigation points brought about by the adoption of WGS-84 as a common geodetic reference frame for international civil aviation. The particular accuracy of the field work values has been based on operational requirements and is in accordance with the provisions set forth in Annexes 11 and 14, Volumes I and II while resolutions requirements are set forth in Annexes 4 and 15. The specified accuracies can, in many cases, easily be exceeded using modern survey instrumentation.

5.1.1 Application of requirements

5.1.1.1 The requirements contained in this chapter apply to all aerodromes/heliports selected by national administrations for international and domestic use and they relate to the surveying, with respect to WGS-84, of the geographical coordinates of navigation elements. Those navigation aids and points whose coordinates contribute directly to air navigation are considered in this chapter as navigation elements. Surveying guidance covers the determination of coordinates, i.e. latitudes and longitudes of certain navigation positions.

5.1.1.2 The results of the surveying of WGS-84-related geographical positions of navigation elements must be reported to the aeronautical information service of the national administration, in accordance with the provisions in Annexes 11 and 14.

5.1.1.3 This chapter includes surveying requirements for those aerodrome/heliport positions, such as designated aircraft stands and navigation checkpoints, whose coordinates may be used for the purpose of checking, calibrating or initializing navigation equipment. The list of navigation elements to be surveyed is given in Table 5-1.

5.2 GENERAL SPECIFICATIONS

5.2.1 The geodetic datum to which coordinates of navigation elements must be referenced is WGS-84. This requirement will be achieved by surveying with respect to an appropriate global geodetic reference frame.

5.2.2 All aeronautical coordinate data which meet the specifications of this manual must be such that their quality can be demonstrated. The accuracy of the field work, with respect to determination of the geographical coordinates of the various navigation elements, has been set in accordance with both current and anticipated operational requirements. Where a navigation aid serves more than one phase of flight and is thereby subject to different operational requirements, the more stringent surveying accuracy requirement must apply.

5.2.3 All position accuracies must relate to a probability of 95% (2 σ) containment unless otherwise stated. Units of measurement must be in accordance with the survey custom and practice of the particular State. All published geographical positions and dimensions must be in accordance with ICAO requirements. In this regard, geographical positions must be published in the form of sexagesimal degrees (degrees, minutes, seconds and decimals of a second) and to the publication resolutions in Annex 15, Appendix 7 and the charting resolutions in Annex 4, Appendix 6.

5.2.4 Dimensions and distances must be quoted in one of the following units:

a) metres (m);

b) feet (1 ft = 0.3048 m); or

c) nautical miles (1 NM = 1 852 m).

5.2.5 Aerodrome/heliport survey control network

In order to determine the position of navigation elements at, and in the vicinity of, designated aerodromes/heliports, a

Table 5-1. Minimum survey accuracy and integrity requirements for navigation elements

Note.— Accuracies are those relative to the established aerodrome/heliport survey control network except where marked by an asterisk (*) when they relate to absolute coordinates with respect to the datum.

Latitude and longitude	*Accuracy data type*	*Integrity classification*
En-route NAVAIDS and fixes, holding, STAR/SID points	100 m surveyed/calculated	1×10^{-5} essential
Aerodrome/heliport reference point	30 m surveyed/calculated	1×10^{-3} routine
NAVAIDS located at the aerodrome/heliport	3 m surveyed	1×10^{-5} essential
Obstacles in the circling area and at the aerodrome/heliport	3 m surveyed	1×10^{-5} essential
Significant obstacles in the approach and take-off area	3 m surveyed	1×10^{-5} essential
Final approach fixes/points and other essential fixes/points comprising instrument approach procedure	3 m surveyed/calculated	1×10^{-5} essential
Runway (landing) threshold	1 m surveyed	1×10^{-8} critical
Runway end (flight path alignment point)	1 m surveyed	1×10^{-8} critical
Runway centre line points	1 m surveyed	1×10^{-8} critical
Taxiway centre line points	0.5 m surveyed	1×10^{-5} essential
Ground taxiway centreline points, air taxiways and transit routes points	0.5 m surveyed/calculated	1×10^{-5} essential
Aircraft standpoints/INS checkpoints	0.5 m surveyed	1×10^{-3} routine
Geometric centre of TLOF or FATO thresholds at heliports	1 m surveyed	1×10^{-8} critical
WGS-84 geoid undulation at aerodrome/heliport elevation position	0.5 m or 1 ft surveyed	1×10^{-5} essential
WGS-84 geoid undulation at runway or FATO threshold, TLOF geometric centre, non-precision approaches	0.5 m or 1 ft surveyed	1×10^{-5} essential
WGS-84 geoid undulation at runway or FATO threshold, TLOF geometric centre, precision approaches	0.25 m or 1 ft surveyed	1×10^{-8} critical
Aerodrome/heliport survey control network (datum transfer)	1 m * surveyed	1×10^{-8} critical

network of survey control stations must be established at each such aerodrome/heliport. The network must consist of a minimum of two inter-visible survey stations at a minimum lateral separation of 500 m. The aerodrome/heliport survey control network may consist of a minimum of four stations so as to provide sufficient redundancy to be able to sustain the loss of one survey station and still enable the orientation to be checked. Survey stations must be strategically located so as to provide maximum utility in subsequent surveys. The monuments of existing aerodrome/heliport survey control networks may be used for the purposes laid down in this chapter.

5.2.6 Control network accuracy requirements

5.2.6.1 The position of each survey station must be determined to an accuracy of 1 m with respect to an appropriate geodetic reference frame. The control network must have an internal accuracy consistent with the need to provide control for the survey of navigation elements to the accuracies set out in the relevant Annexes and in this chapter. The aerodrome/heliport survey control network may have an internal consistency of better than 10 cm.

5.2.6.2 Mathematical transformation methods based on a single set of average transformation parameters, which relate known (existing) datum to WGS-84, must not be used for the purpose of determining the coordinates of the aerodrome/heliport survey control network.

5.2.7 Monumentation of survey control stations

The survey control stations must consist of standard types of survey monuments (see Attachment A to this chapter). Different types of monuments will be appropriate for different locations and ground conditions at the aerodrome/heliport; it is the surveyor's responsibility, under the guidance of the national administration, to decide on the most appropriate type. Investigation may be made prior to the installation of a survey monument to ensure that underground cables and services are not affected by the installation. Where the survey network consists of fewer than the recommended four stations, monuments of a larger size must be used.

5.2.8 Station numbering

5.2.8.1 Each survey station must carry an individual number. This will ensure that, where a station has been destroyed and subsequently replaced by a new station in approximately the same location, misidentification does not occur.

5.2.8.2 Station labelling and numbering must be such that there is no doubt as to the provenance or identity of the survey station. Uniform labels (e.g. stamped disks) may be used at individual aerodromes/heliports for all survey stations. An unambiguous numbering system, identifying the aerodrome/heliport, year and station number, may be used (see Attachment A to this chapter). Where an existing, substantial topographic surface feature is used as a survey station, the station must be clearly marked with durable paint.

5.2.9 Station location plan

An aerodrome/heliport survey network plan, at a scale of 1:2 000 or other appropriate standard cartographic scale, indicating the location of all survey stations and principal topographic features must be prepared. The plan may be orientated to true north or, alternatively, have the direction of true north indicated on the plan.

5.2.10 Station descriptions

5.2.10.1 A comprehensive aerodrome/heliport survey network station description must be prepared. This must consist of a written description and a clear diagram indicating the dimensions and direction indicators to other visible points on the aerodrome/heliport survey control network.

5.2.10.2 A photograph of the station showing background detail may be included in the description. Inspections may be made to check on the general condition of the aerodrome/heliport survey network, and any disturbance or damage must be recorded.

5.2.11 Determination of control coordinates

5.2.11.1 One of the following methods of coordinate determination must be used to fix the positions of the aerodrome/heliport survey control network.

a) *Direct geodetic connection*. Survey measurements must be taken to connect the aerodrome/heliport survey control network to a national or international (e.g. ITRF) geodetic frame in such a way that the survey error in the connection does not contribute significantly to the coordinate error of the aerodrome/heliport survey control network. This is the preferred option, in that it consists of the most accurate method of observation and incorporates a directly observed connection to the approved geodetic reference frame. Static differential GPS connections may be made to, preferably, three points on an appropriate geodetic network but, in all cases, must be made to a minimum of two.

b) *Derived geodetic connection*. Where the local relationship between the existing geodetic control network and WGS-84 is known to an accuracy commensurate with the requirements in this manual, then standard, nationally or regionally approved transformation methods may be used to determine the coordinates of an existing aerodrome/heliport survey control network. Where this method is adopted, a full description of the transformation method and the values of the transformation parameters must be included in the report. Full details of the connection of the existing aerodrome/heliport survey control network to the existing geodetic network must be included in the survey report (an existing network means one that existed at the aerodrome/heliport prior to the implementation of WGS-84 at that aerodrome/ heliport).

c) *Direct observation of WGS-84*. For those regions where national or international coordinates are unavailable, the coordinates of the aerodrome/heliport survey control network must be determined by direct observation of WGS-84 by using an appropriate type of geodetic GPS receiver. All such observations must be controlled by simultaneous observations made at points of known absolute WGS-84 coordinates. The observation and computation method must be such that the absolute coordinates of the aerodrome/heliport survey control network are determined to the accuracy stated in this chapter.

5.2.12 Determination of the local relationship between the known existing datum and WGS-84

5.2.12.1 Where existing relative surveys need to be related to WGS-84 (e.g. aerodrome/heliport obstacle surveys), observations must be taken to determine the local relationship (difference in latitude, longitude, orientation and scale) between the known existing datum and WGS-84, except where the required information is provided by a derived geodetic connection.

5.2.12.2 Where used, the local relationship between the known existing geodetic datum and WGS-84 must be determined to an accuracy commensurate with the relative accuracy of the data to be transformed. The values and accuracies of the local relationship must be declared in the survey report.

5.2.13 Report requirements

5.2.13.1 All survey work undertaken to determine the coordinates of navigation facilities at aerodromes/heliports must be reported in the format specified in Attachment C to this chapter. Where an existing national reporting practice differs from that shown in this chapter, the national administration may make a case in support of the national practice, where the national practice can be shown to be compatible.

5.2.13.2 Table 5-1 lays down the minimum survey accuracies that must be achieved. These accuracies can, in many cases, easily be exceeded using modern survey instrumentation. All survey observations may be made and recorded to the resolution and accuracy of the equipment used so that future requirements for surveys of greater precision might be met. Where surveys are undertaken using equipment or techniques that yield height data as well as horizontal position, these must be comprehensively recorded and included in the survey report.

5.3 SURVEY REQUIREMENTS FOR AERODROME/HELIPORT-RELATED NAVIGATION ELEMENTS

5.3.1 Runway centre lines and thresholds

5.3.1.1 For surveying purposes, the centre line of a runway must be taken as being the geometric centre of the width of the bearing surface, this definition taking precedence over any existing runway centre line markings or lighting. Where the edge of the runway is irregular or connected to a taxiway, an appropriate theoretical line must be selected which best identifies the probable edge of the runway.

5.3.1.2 For surveying purposes, threshold positions must be taken as being at the geometric centre of the runway and at the beginning of the paved surface, i.e. the beginning of that portion of the runway usable for landing. Where thresholds are marked by appropriate threshold markings (e.g. displaced thresholds), these must be taken as the threshold points. Where threshold lighting is surveyed, the locations must be described on the diagram accompanying the report. Where there is no threshold lighting, the surveyor must select an appropriate point for survey in accordance with the diagrams shown in Attachment B to this chapter.

5.3.1.3 If the runway has only one threshold certified for landing, the runway end position must be surveyed. For surveying purposes, the runway end position (flight path alignment point) must be taken as being at the geometric centre of the runway and at the end of the paved surface, i.e. the end of that portion of the runway usable for landing.

5.3.1.4 Survey witness marks must be installed to enable the threshold survey point to be re-established in the event of resurfacing, repainting or verification. In addition, two associated runway centre line points, at a separation of not less than 10% of the runway length, must be surveyed. The surveyor must, in processing the survey data, determine and report on the collinearity of the three points. Where a runway has a threshold at each end, then the two thresholds and the two additional runway centre line points may be surveyed, the collinearity then being determined for the group of four points.

5.3.2 Derived threshold coordinates

5.3.2.1 Where a point has been selected for survey which is not coincident with the runway threshold but offset along the centre line, then the coordinates of the threshold must be determined by the national administration. One method of calculation for this task is shown in Attachment D to this chapter.

5.3.2.2 The newly derived threshold coordinates must be submitted to the same collinearity check as specified in 5.3.1.4.

5.3.3 Taxiways and stand/checkpoints

5.3.3.1 *General guidelines*

5.3.3.1.1 In accordance with the operational requirements for advanced surface movement guidance and control systems (A-SMGCS), pilots should be provided with continuous guidance and control during the landing roll-out, taxiing to the parking position and from the parking position to the runway holding point to line up for an appointed take-off position, and during the take-off roll.

5.3.3.1.2 Since determination of the appropriate taxiway centre line points and aircraft stand points for pilot guidance and control are predicated on the aerodrome/ heliport surface markings, application of the following paragraphs must be restricted to those aerodromes/heliports that conform to the SARPs for markings included in Annex 14, Volume I, 5.2 and Volume II, 5.2.11 and 5.2.12.

5.3.3.1.3 Diagrams depicting appropriate points on aerodrome/heliport movement areas are shown in Attachment B to this chapter.

5.3.3.1.4 Except as provided in 5.3.3.2.1, for surveying purposes the centre (mid-width) of the taxiway centre line marking, apron taxilane marking or the aircraft stand guide line marking must be taken as the reference datums. Survey witness marks must be installed to enable the taxiway centre line, apron taxilane and aircraft stand guide line marking survey points to be re-established in the event of resurfacing or repainting, and for verification purposes.

5.3.3.1.5 The points of commencement and ends of straight sections of taxiways, apron taxilanes and aircraft stand point guidance lines markings must be surveyed. Sufficient additional points must be surveyed to maintain the required accuracy along the lines.

5.3.3.1.6 For curved sections of taxiways, apron taxilanes and aircraft stand guide line markings, the commencement and end of the curved section centre line must be surveyed together with the position of the centre point of the arc and its radius. In the case of a compound curve, the centre and radius of each arc and the commencement and end of each of the arcs must be surveyed. Where this is impracticable in the field, a series of sequential points must be surveyed along the curved section of the centre line with a maximum arc to chord distance not exceeding 0.25 m for taxiways and 0.10 m for apron taxilanes and aircraft stand guide line markings. Sufficient points must be surveyed to maintain the required accuracy along the lines. The surveyor must, in processing the data, conduct a graphical inspection of the survey points to ensure collinearity.

5.3.3.2 *Taxiways*

5.3.3.2.1 Annex 14, Volume I, 5.2.8.5 and Figure 5-6 recommend that taxiway markings on the runway are offset by 0.9 m parallel to the runway centre line and associated runway centre line marking to enable pilots to visually distinguish between the runway centre line markings and taxiway markings for exits from the runway. However, to permit uninterrupted transition from the actual runway centre line to the taxiway centre line and to provide the required continuity of guidance for the aircraft navigation data base, differentiation must be made between the surface markings and the actual path the aircraft must follow. Therefore, for the guidance of aircraft entering or exiting the runway for take-off or landing, the following must be surveyed:

a) the point at which the radius of turn, prescribed by the appropriate authority for each taxiway, is tangential to the runway centre line (as determined under 5.3.1.1) and the point at which that radius of turn joins the taxiway centre line marking at a tangent;

b) the point that prescribes the centre of the arc; and

c) the radius of the arc.

Where this is impracticable in the field, a series of sequential points must be surveyed along the curved section of the centre line of taxiways.

5.3.3.2.2 Where taxiway centre line marking is provided on a runway that is part of a standard taxi route, or a taxiway centre line is not coincident with runway centre line, the following points must be surveyed:

a) the point on the taxiway marking at which the taxiway enters the runway;

b) the points at which the taxiway deviates from a straight line;

c) the intersection of the taxiway centre line marking and boundary of each “block” that has been published as part of the airport movement and guidance control system; and

d) the point on the taxiway marking at which the taxiway exits the runway.

5.3.3.2.3 In defining taxiways, the following points must be surveyed at the centre of the centre line marking of each taxiway, as appropriate:

a) intermediate holding positions and runway holding positions (including those associated with the intersection of a runway with another runway when the former runway is part of a standard taxi route) and for points established for the protection of sensitive areas for radio navigation aids;

b) taxiway intersection markings;

c) intersection of other taxiways, including taxiways described in paragraph 5.3.3.2.2;

d) intersections of "blocks" defined for surface movement, guidance and control systems;

e) commencement and end of selectable taxiway lighting systems provided as part of the surface movement, guidance and control systems, where different from subparagraph d) above; and

f) at stop bars.

5.3.3.2.4 In defining a helicopter air taxiway, the centre of each air taxiway marker must be surveyed, as appropriate.

5.3.3.3 *Aircraft standpoints*

5.3.3.3.1 In defining the aircraft stands, the following points must be surveyed at the centre of the guide line marking of the aircraft stands, as appropriate:

a) taxilane centre lines;

b) lead-in line(s);

c) turning line;

d) straight section of the turning line;

e) nosewheel stopping position;

f) true heading of the alignment bar; and

g) lead-out line(s).

5.3.3.3.2 Where aircraft stands are utilized by more than one aircraft type and different guide line markings exist, a diagram must be prepared by the surveyor showing the arrangement of the markings in use, together with an indication of the points surveyed. Where all the stands at an aerodrome/heliport are marked uniformly, only a single diagram need be prepared.

5.3.4 Navigation checkpoints

Where navigation checkpoints used for the validation of navigation systems are surveyed, their coordinates must be determined to the accuracy laid down in Table 5-1. Where these checkpoints coincide with aircraft stands, the nosewheel stopping position must be surveyed in accordance with 5.3.3.3.

5.3.5 Road holding positions

In accordance with local requirements, significant points shall be surveyed to meet the needs of the surface movement guidance and control system for vehicular traffic on the movement area of the aerodrome.

5.3.6 All other aerodrome/heliport navigation elements

For all other aerodrome/heliport navigation elements that require surveying, the geometric centre of the element must be surveyed except where a different specific survey point is standardized for the element, as indicated in Attachment B to this chapter.

5.4 AERODROME/HELIPORT SURVEY REPORT REQUIREMENTS

All survey work undertaken to determine the coordinates of navigation elements at aerodromes/heliports must be reported in the format laid out in Attachment C to this chapter. Where an existing national reporting practice differs from that shown in this chapter, the national administration may make a case in support of the national practice, where the national practice can be shown to be compatible.

5.5 SURVEY REQUIREMENTS FOR NAVIGATION AIDS

5.5.1 En-route and aerodrome/heliport navigation aids

5.5.1.1 The coordinates of en-route and aerodrome/heliport navigation aids must meet the accuracy requirements specified in Table 5-1. Where existing coordinates of navigation aids that meet the accuracy and integrity requirements are converted to WGS-84 mathematically, the conversion process must be shown to be such that the required coordinate accuracies are maintained.

5.5.1.2 Where the quality (for surveying purposes this relates to accuracy and integrity requirements) of existing coordinates cannot be determined, they must be surveyed to the accuracy specified in Table 5-1. In all cases, surveyed coordinates may be published in preference to coordinates determined by graphical methods.

5.5.2 Description of en-route and aerodrome/heliport navigation aids

The diagrams of the most common en-route and aerodrome/heliport navigation aids are shown in Attachment B to this chapter. For navigation aids not described in Attachment B, the position of the geometric centre of the antenna must be surveyed. Where collocated VOR/DME are surveyed, the position of the DME element must be taken as the position for both systems.

5.6 EN-ROUTE SURVEY REPORT REQUIREMENTS

All survey work undertaken to determine the coordinates of en-route navigation elements must be reported in the format specified in Attachment C to this chapter. Where an existing national reporting practice differs from that shown in this chapter, the national administration may make a case in support of the national practice, where the national practice can be shown to be compatible.

5.7 USE OF SOFTWARE

Where software is used for any of the survey processing, it must be demonstrated that it functions correctly. This demonstration must take the form of a written report showing that the software produces the same results as standard computation.

5.8 DIGITAL FORMAT FOR THE DELIVERY OF SURVEY DATA

A digital format for the delivery of survey data is included in Chapter 7. This format may be used for the delivery in digital form of all survey data. Where a national standard predates the format in Chapter 7 or where the State prefers its own format, the relationship between the two formats must be described in detail.

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Attachment A. MONUMENTATION

1. GENERAL

Where survey markers are installed, they must be of a type appropriate for the task and for the surface and ground type in which they are installed. Designs of suggested survey markers are shown in Figures 5A-1, 5A-2 and 5A-3, but other types of markers may be equally appropriate.

2. NUMBERING SYSTEM FOR SURVEY MARKERS

2.1 Each survey control point that forms part of the aerodrome/heliport survey control network may be marked in the field with a unique identification number.

2.2 The system of numbering may include the aerodrome/heliport identifier, the station identifier and the year of establishment. Although the aerodrome/heliport identifier will be the same for each station at that aerodrome/heliport and, therefore, will serve no local purpose, its inclusion is important for identification purposes in digital databases.

2.3 Station identifiers, whether alphabetic or numeric, may be assigned chronologically with the construction of the station. The inclusion of the year allows the time of establishment to be referenced and lessens confusion where replacement stations have been established. Alternatively, a simple consecutive numbering system can be used.

2.4 While numbering systems will vary from State to State, it is important that each system include a means whereby the stations are not confused with other survey markers being established at the same aerodrome/heliport. A simple consecutive numbering system alone, without other identifiers, would not be suitable.

Example:

Aerodrome/heliport ICAO location indicator

Station identifier

Year of establishment

LYBE — C — 2001

FIGURES FOR ATTACHMENT A

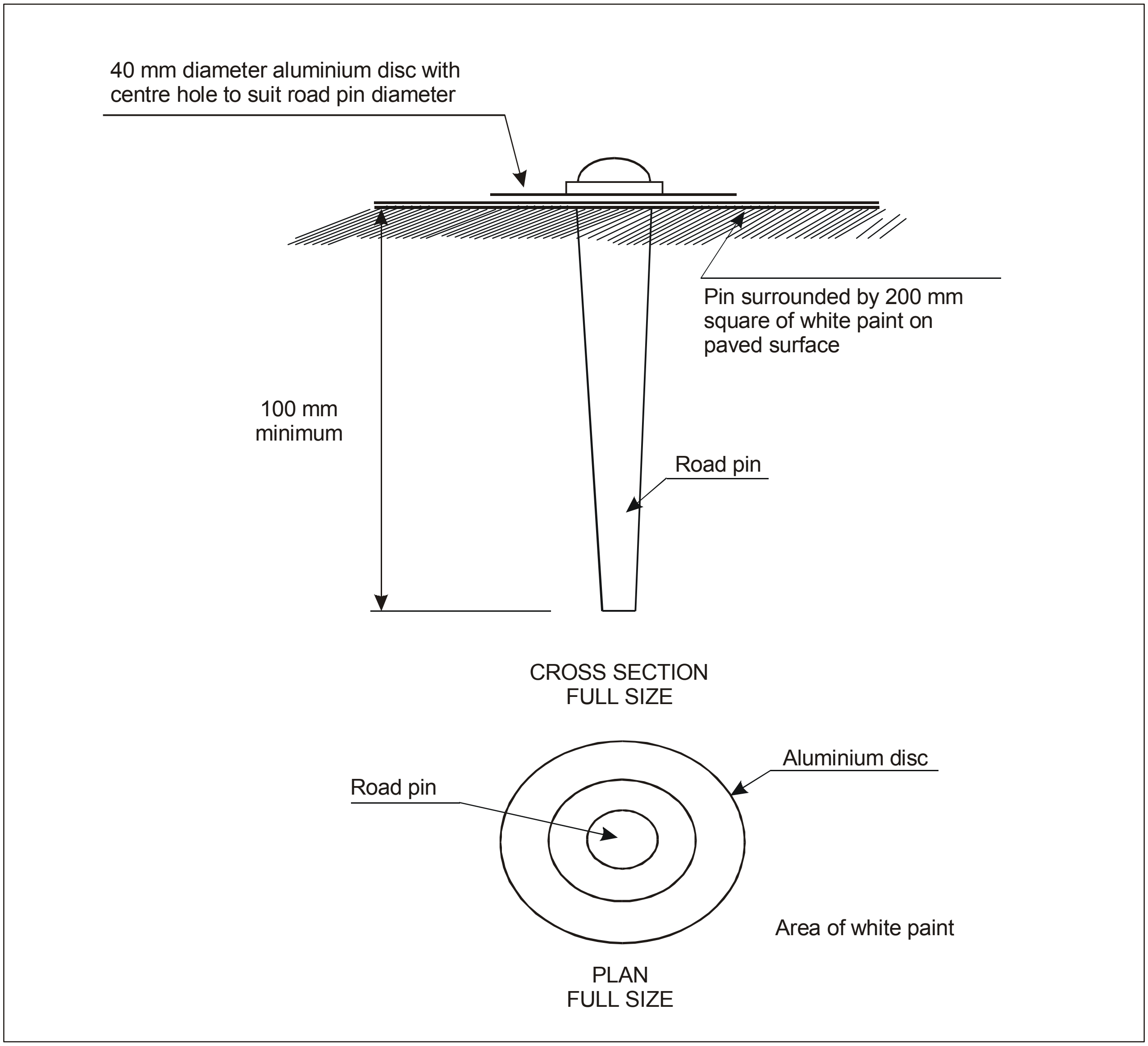

Figure 5A-1. Survey marker

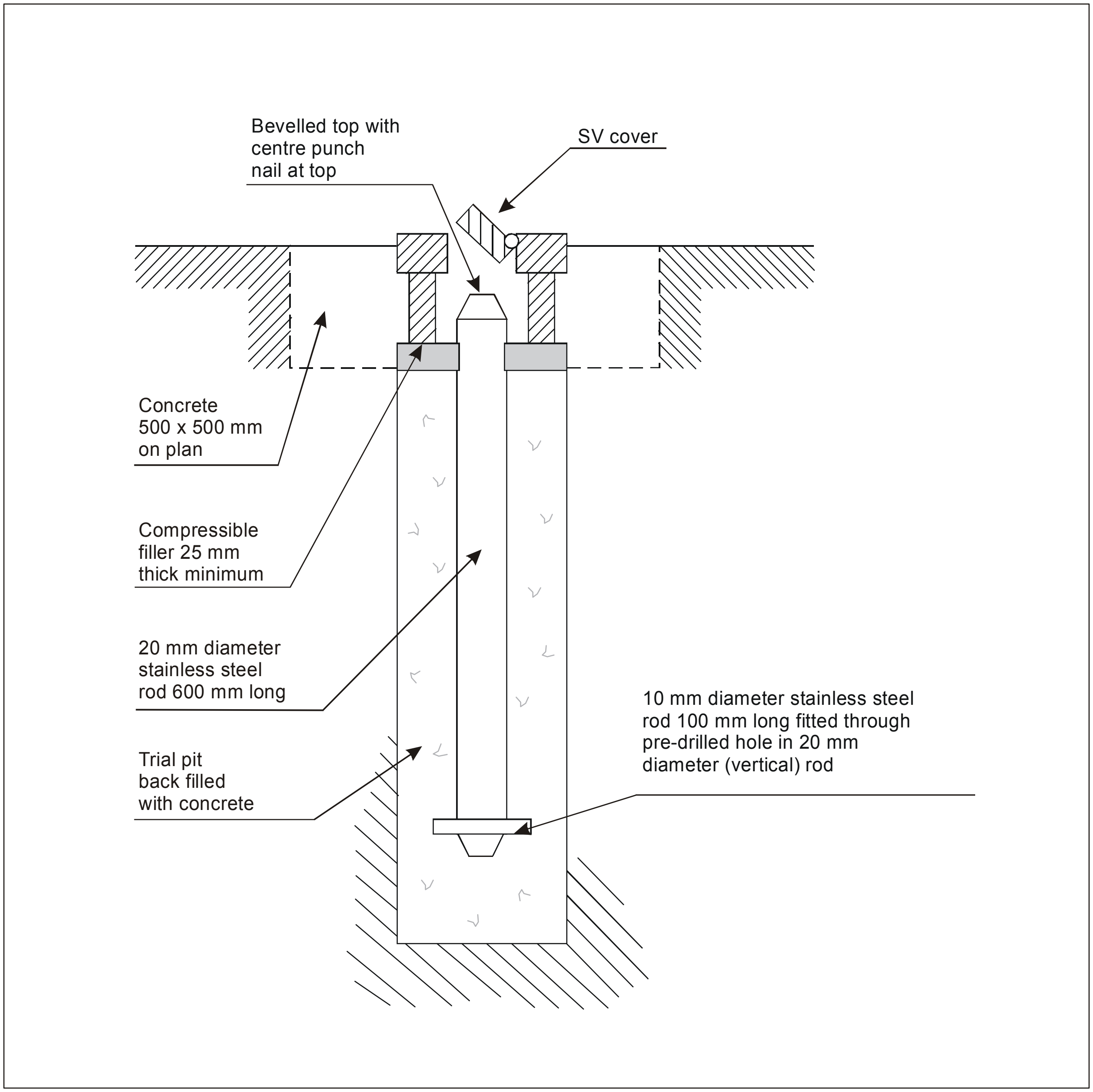

Figure 5A-2. Survey marker

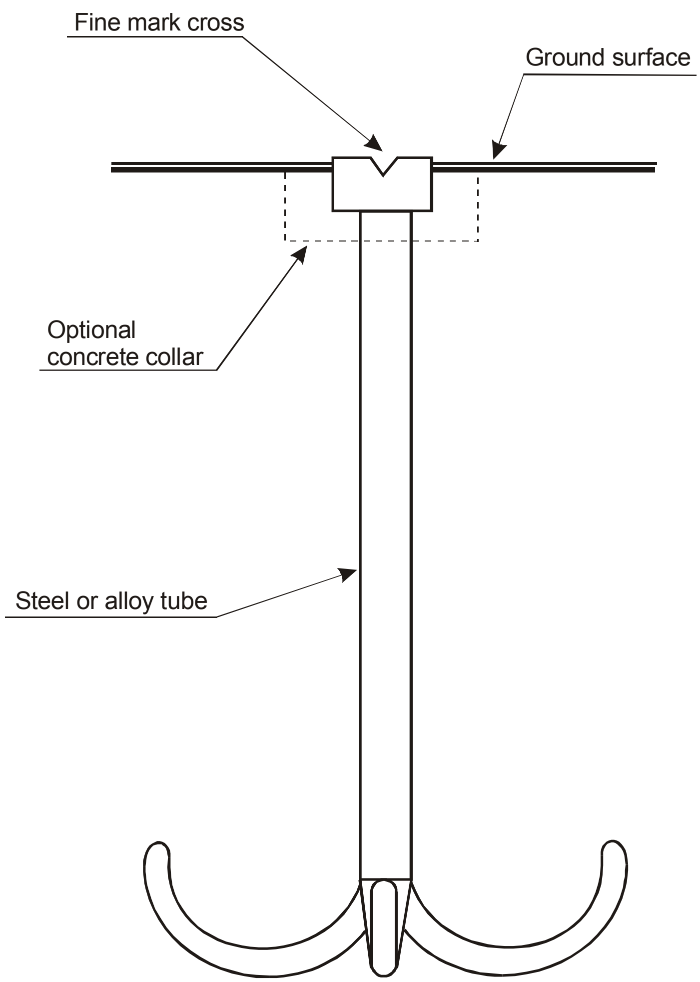

Figure 5A-3. Survey marker

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Attachment B. DESCRIPTION OF GEOGRAPHICAL POSITIONS

In order to standardize the determination of threshold positions for survey purposes, the following guidance is provided:

a) diagrams most closely representing the runway markings may be selected and used as the basis for a description of the location of the markings (lights) and the positions selected for survey;

b) where none of the diagrams in this attachment are appropriate, a new diagram must be prepared, showing the actual arrangement of the markings and the positions selected for survey;

Note.— Wing-bar threshold lights and lights installed ahead of the runway hard surface have no direct survey status with respect to thresholds.

c) where existing national standards are used, the survey report must indicate the equivalence to the diagrams shown in this attachment.

FIGURES FOR ATTACHMENT B

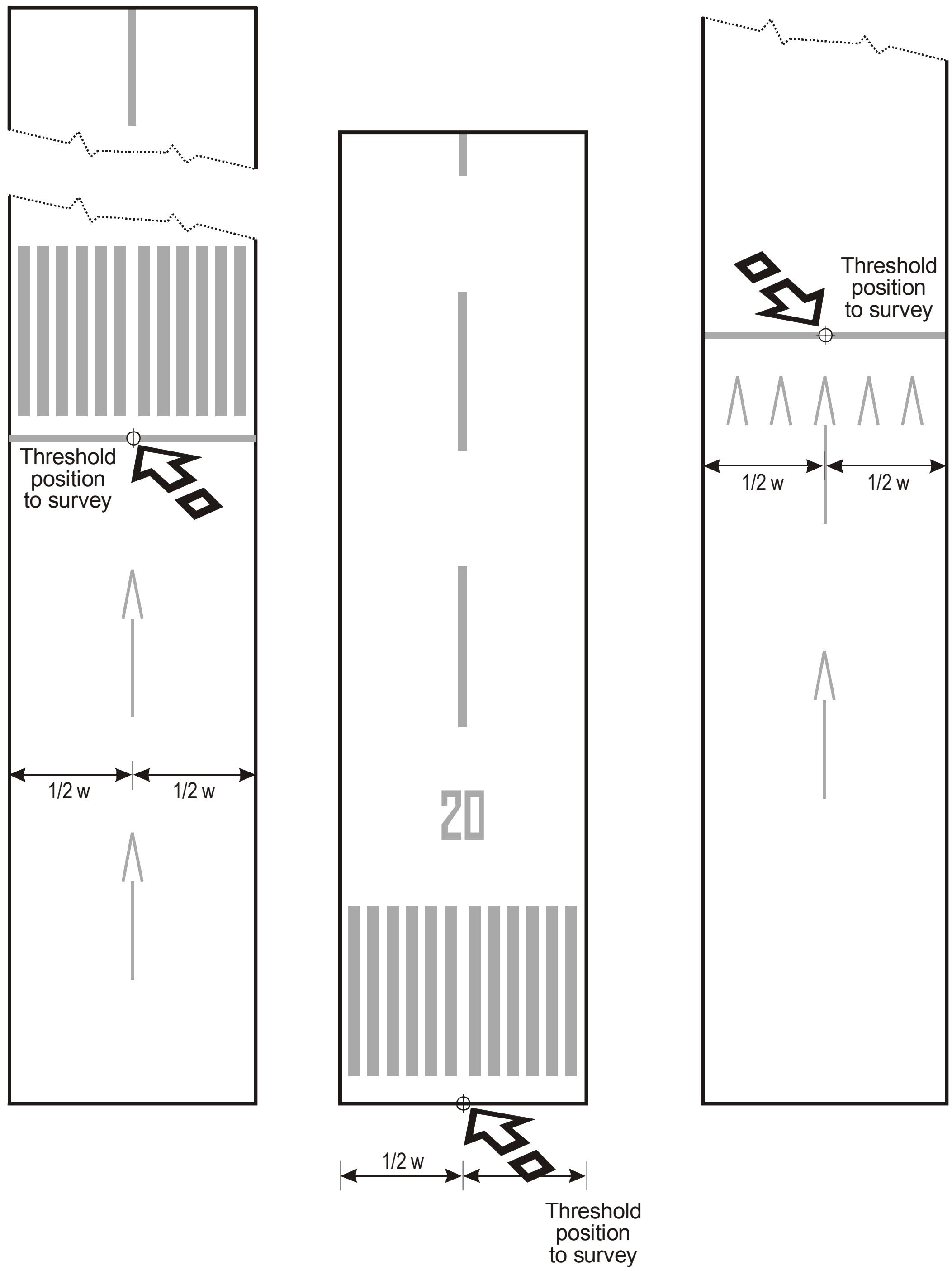

Figure 5B-1. Threshold planimetric position to be surveyed

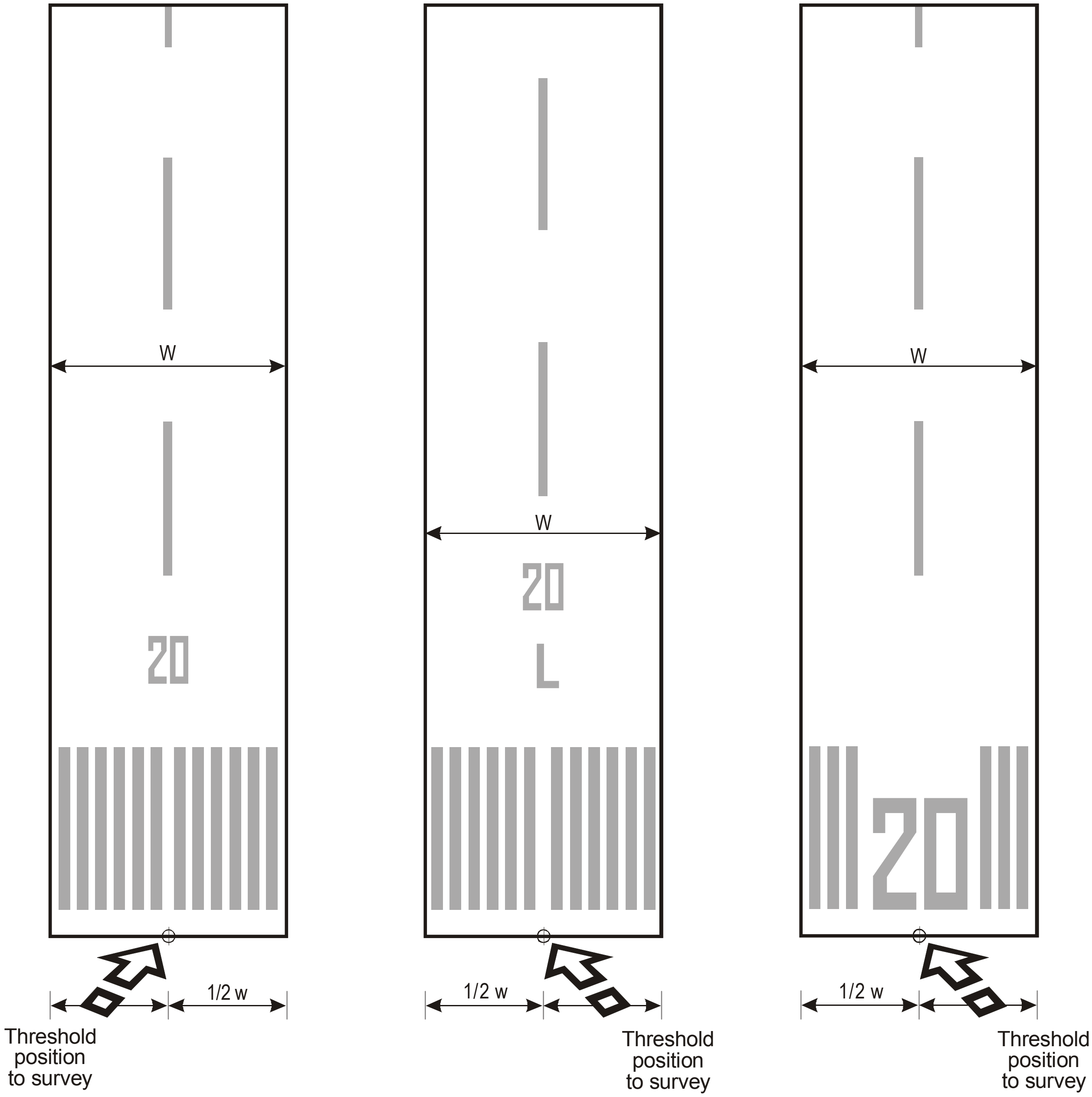

Figure 5B-2. Threshold planimetric position to be surveyed

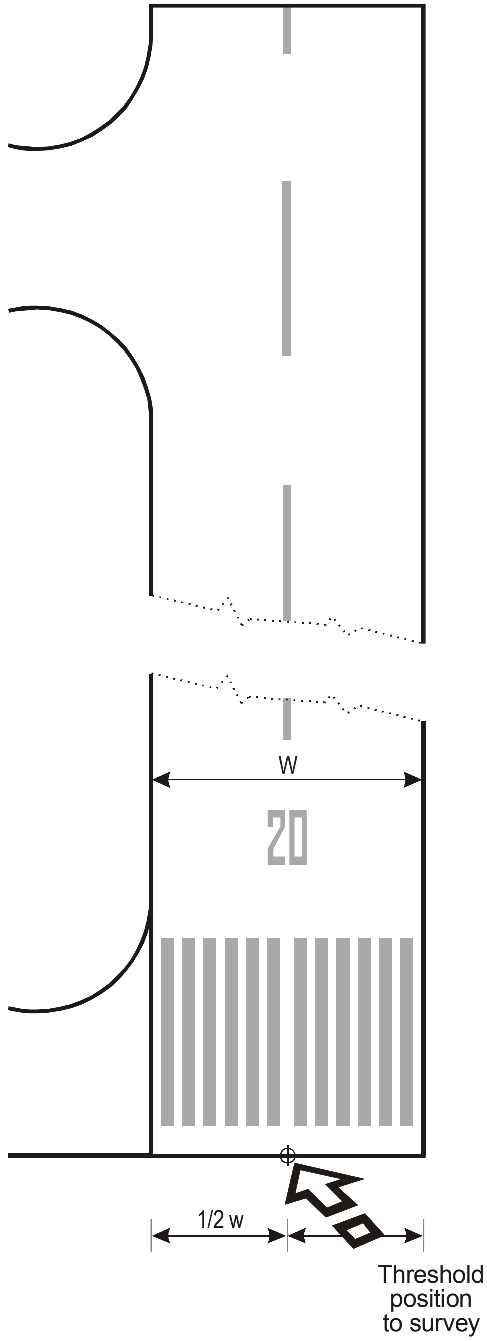

Figure 5B-3. Threshold planimetric position to be surveyed

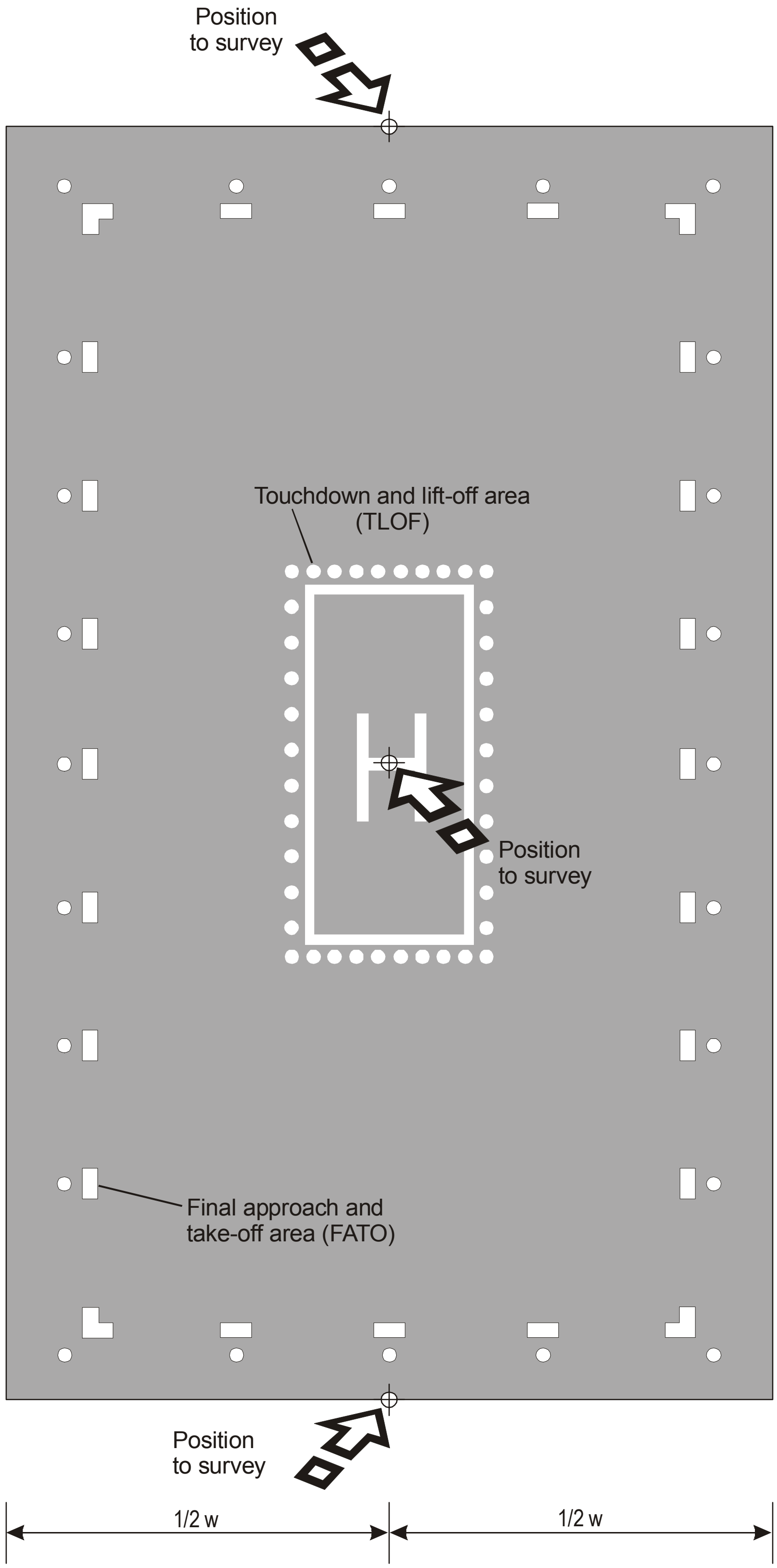

Figure 5B-4. TLOF and FATO planimetric threshold positions to be surveyed

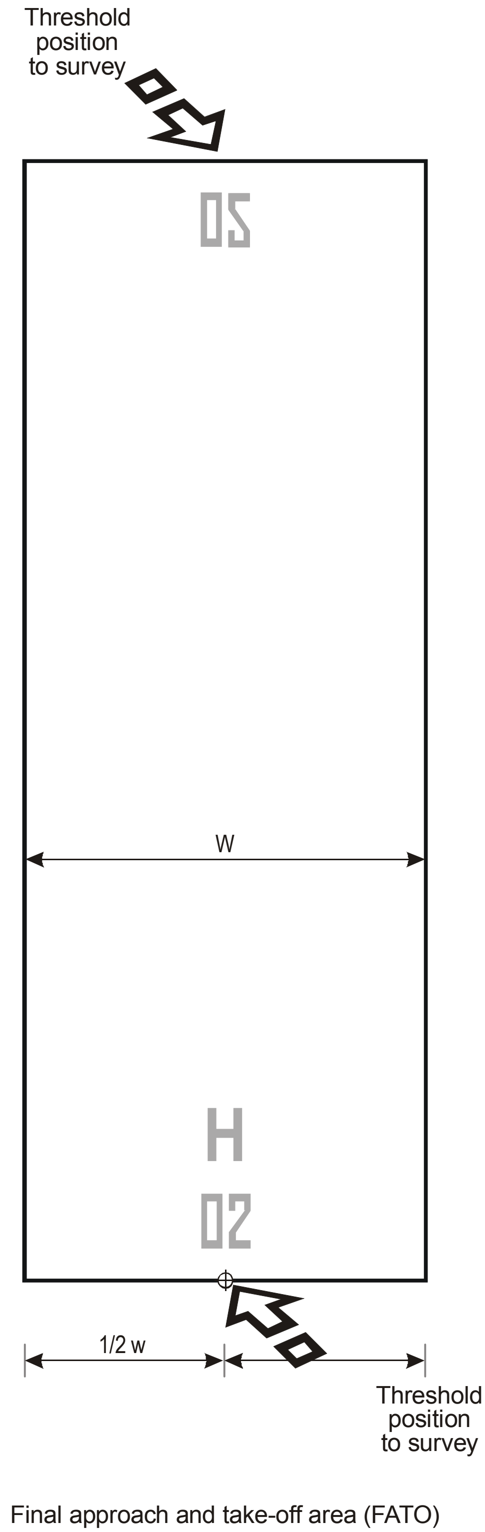

Figure 5B-5. FATO planimetric threshold position to be surveyed

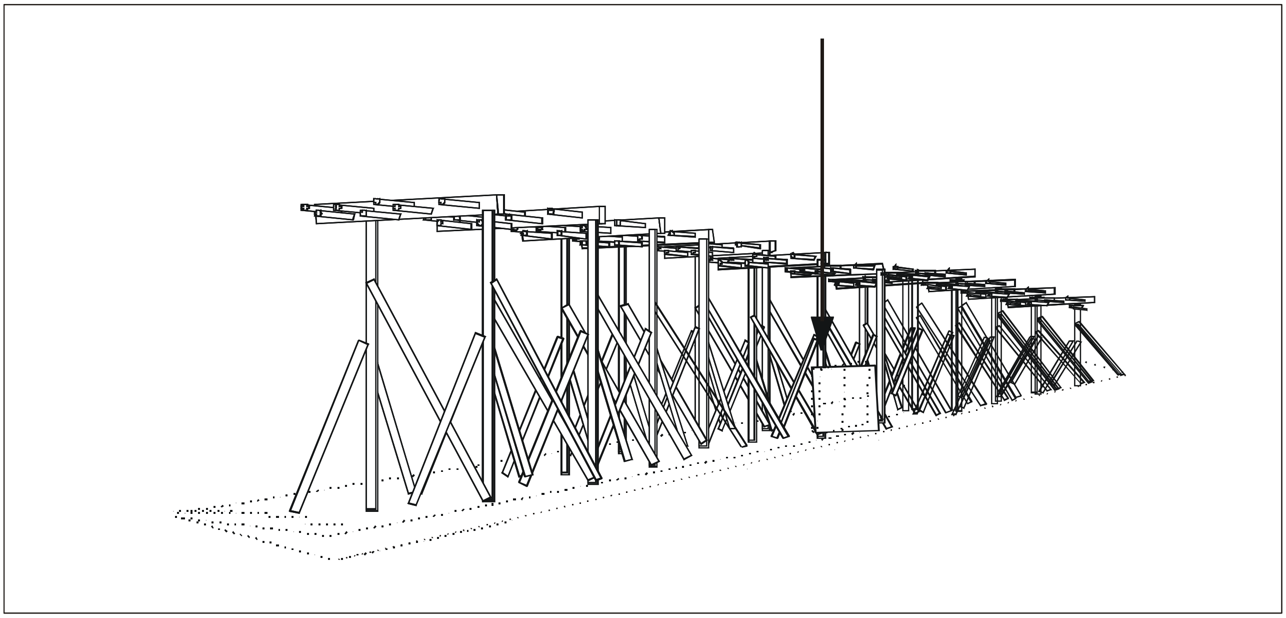

Figure 5B-6. ILS localizer

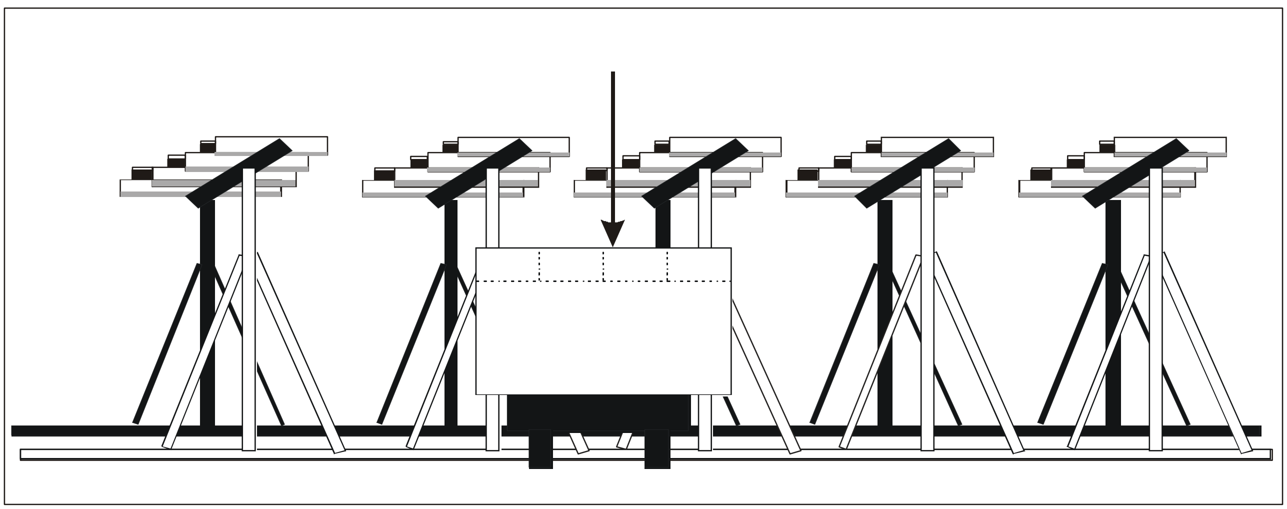

Figure 5B-7. MLS

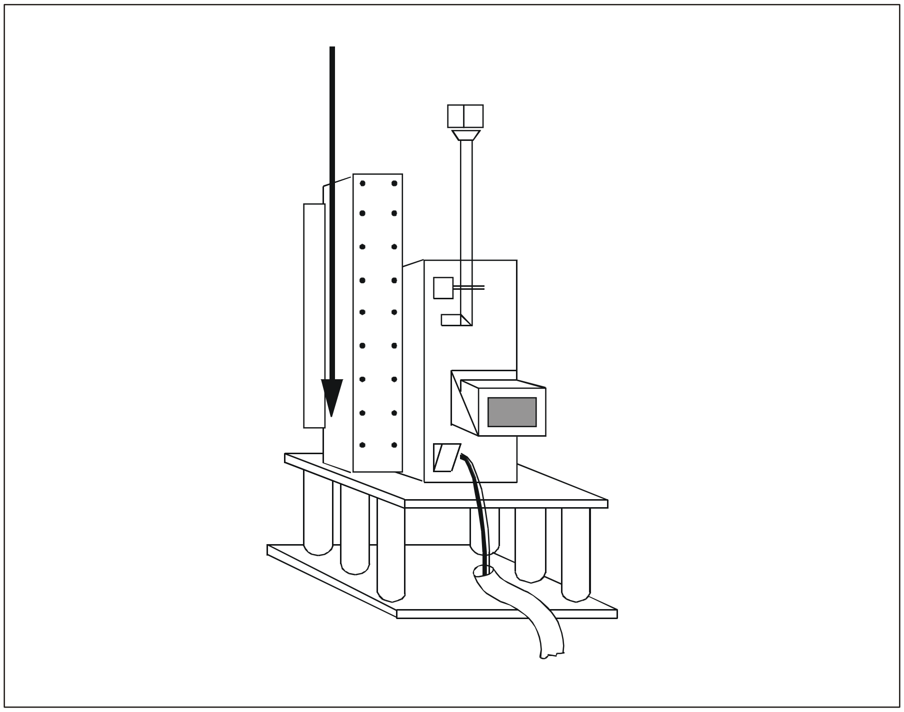

Figure 5B-8. MLS

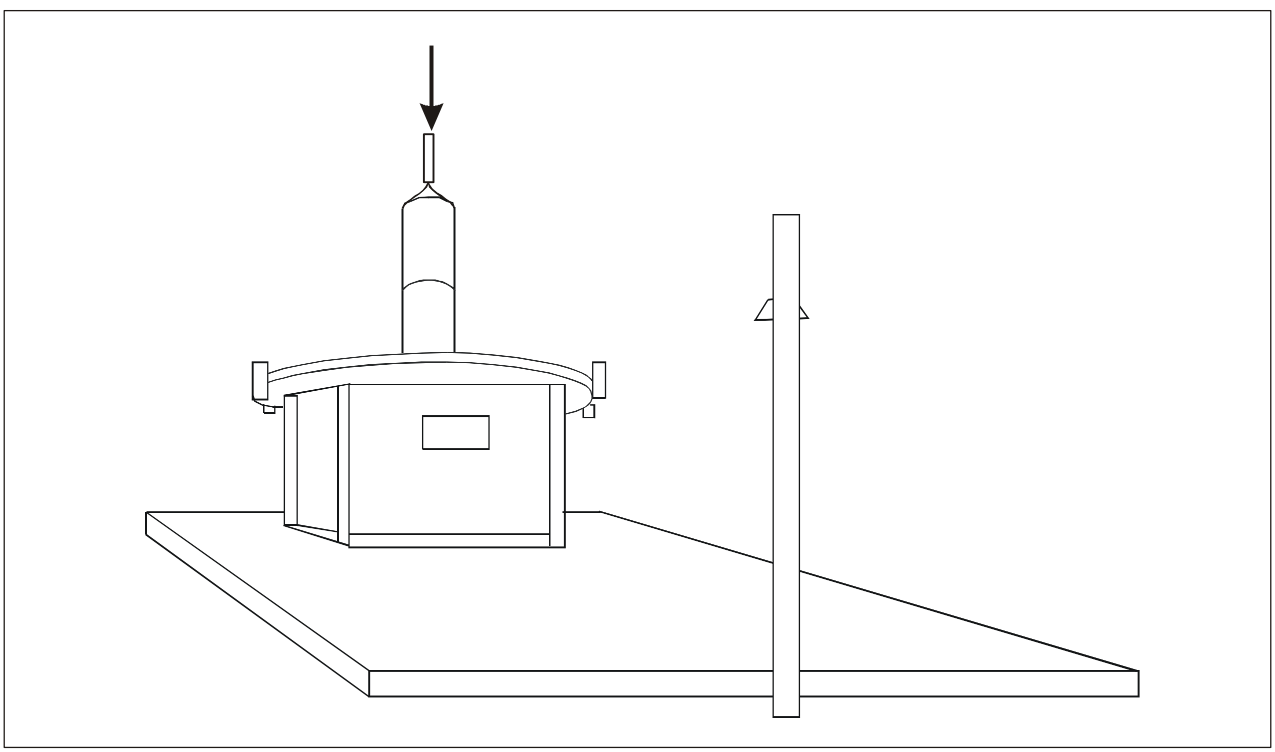

Figure 5B-9. VOR/DME

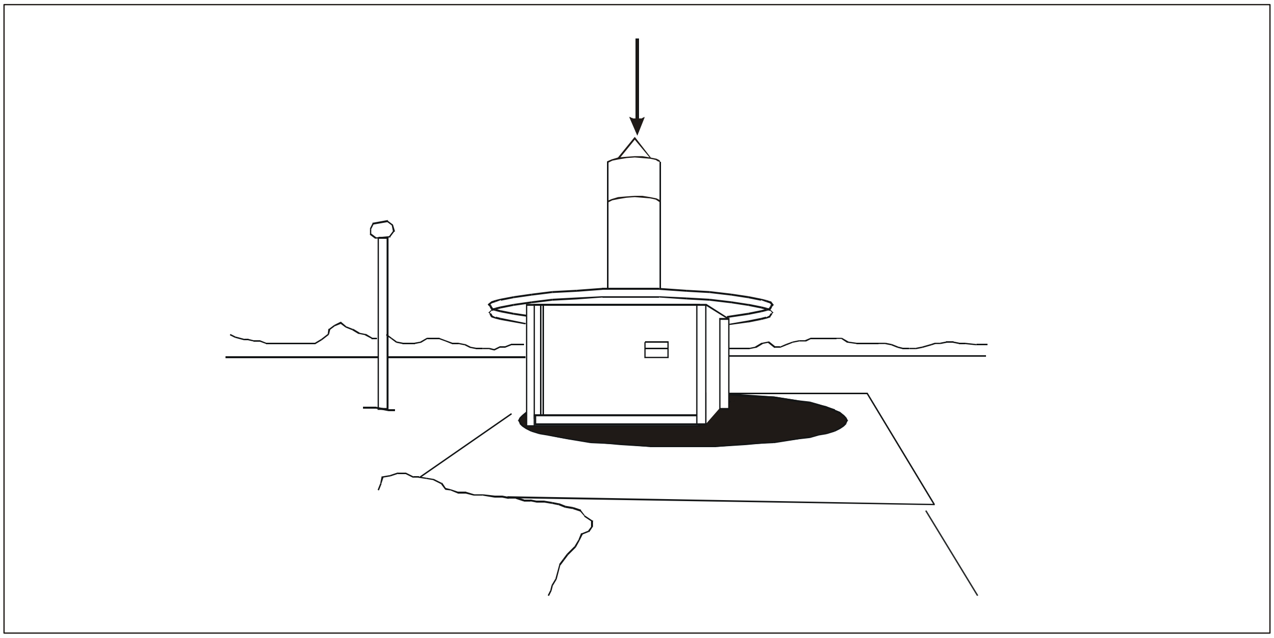

Figure 5B-10. VOR

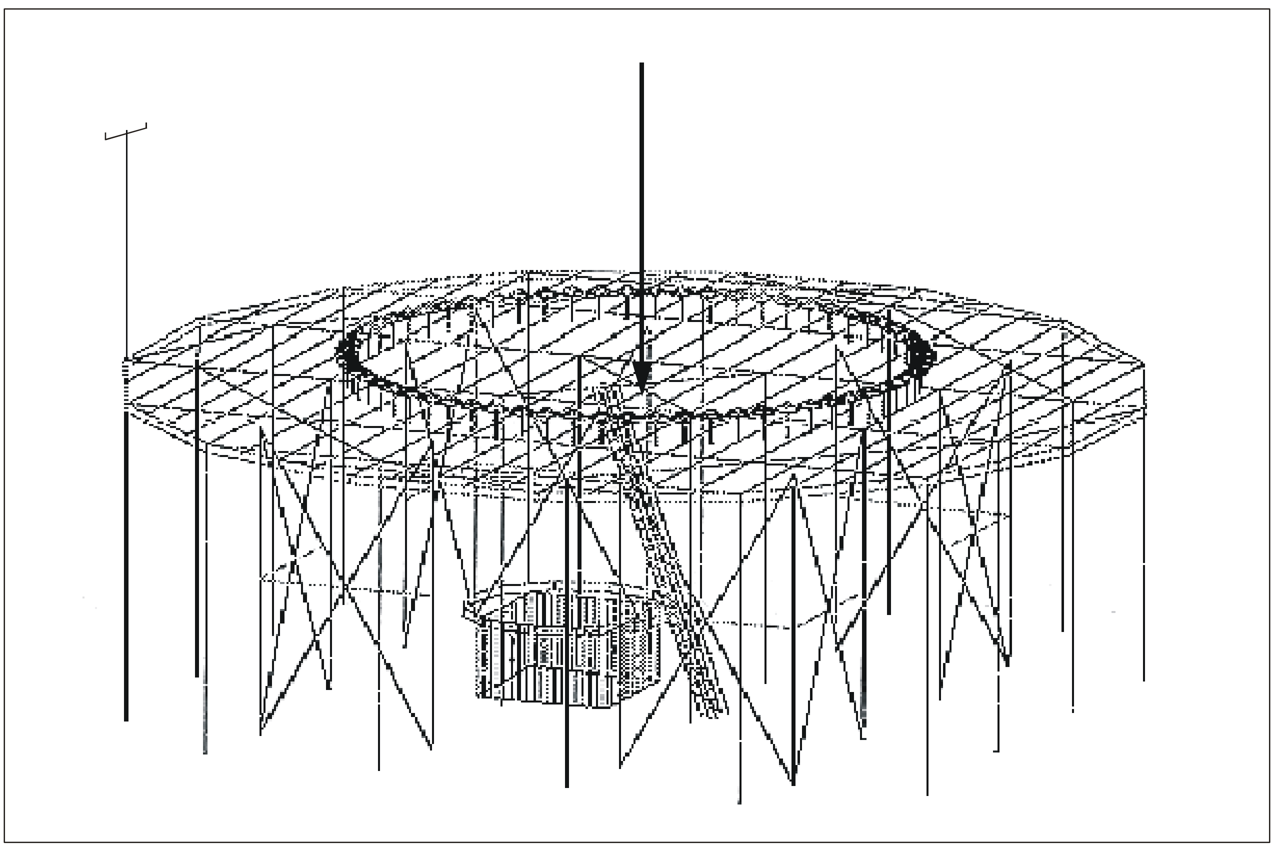

Figure 5B-11. DVOR/DME

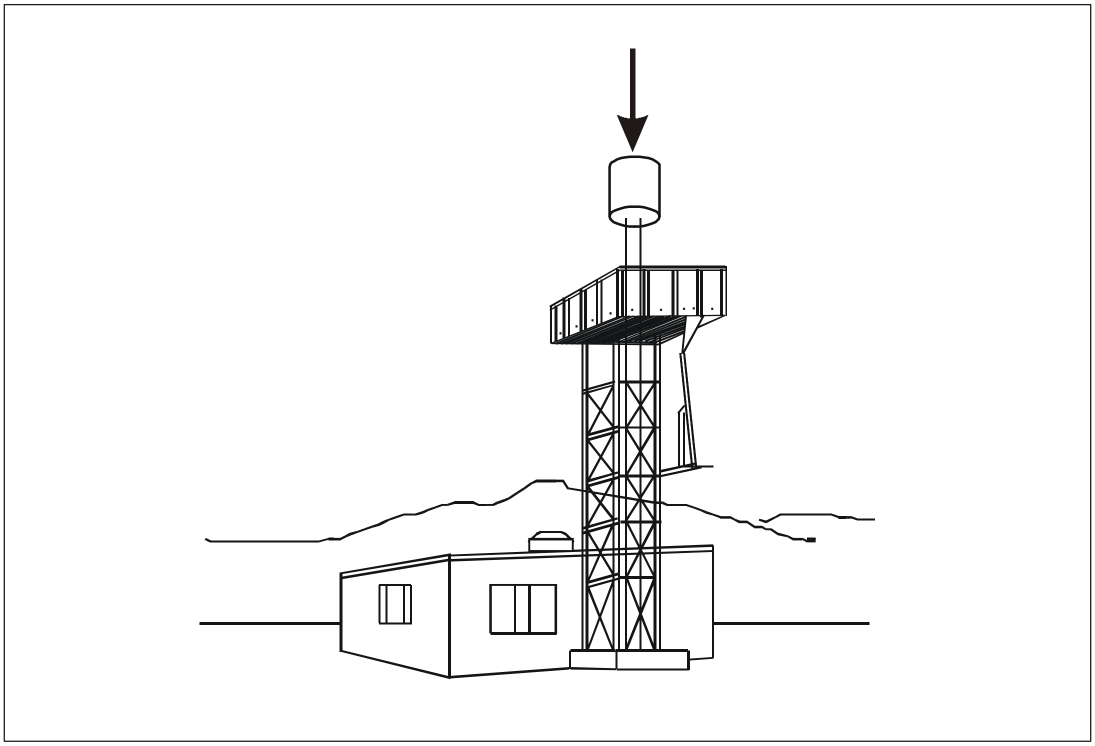

Figure 5B-12. TACAN

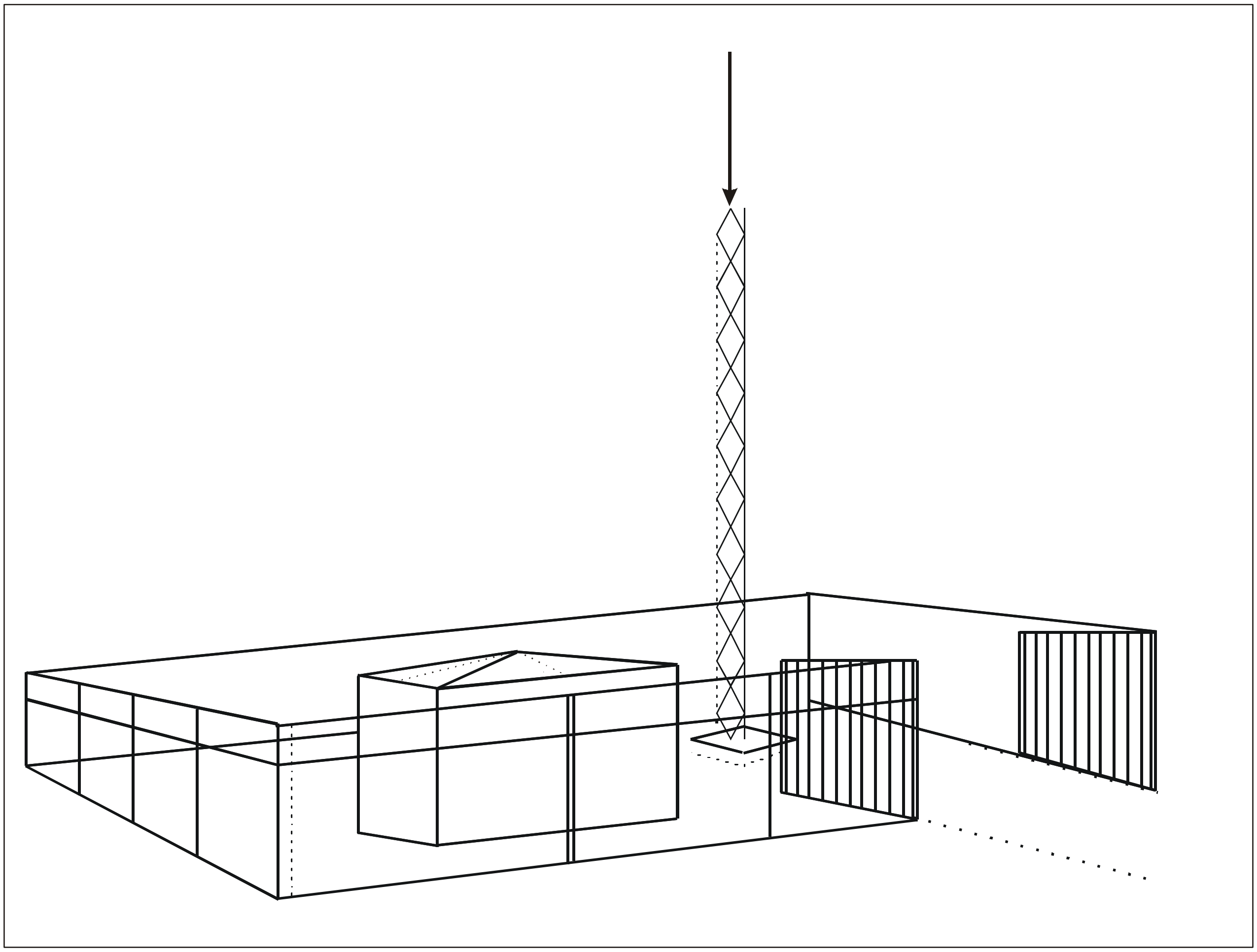

Figure 5B-13. NDB

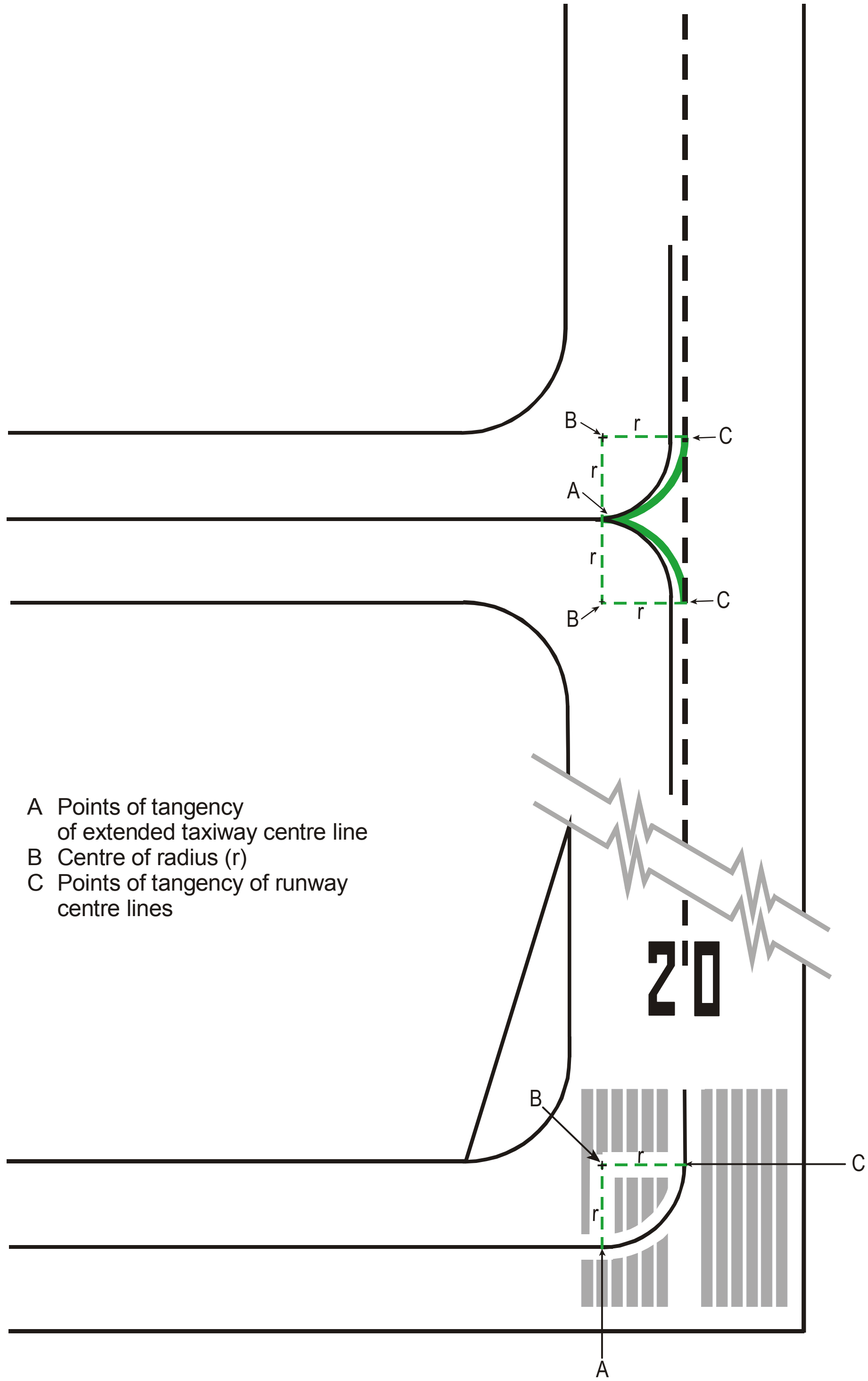

Figure 5B-14. Runway and taxiway intersections to be surveyed

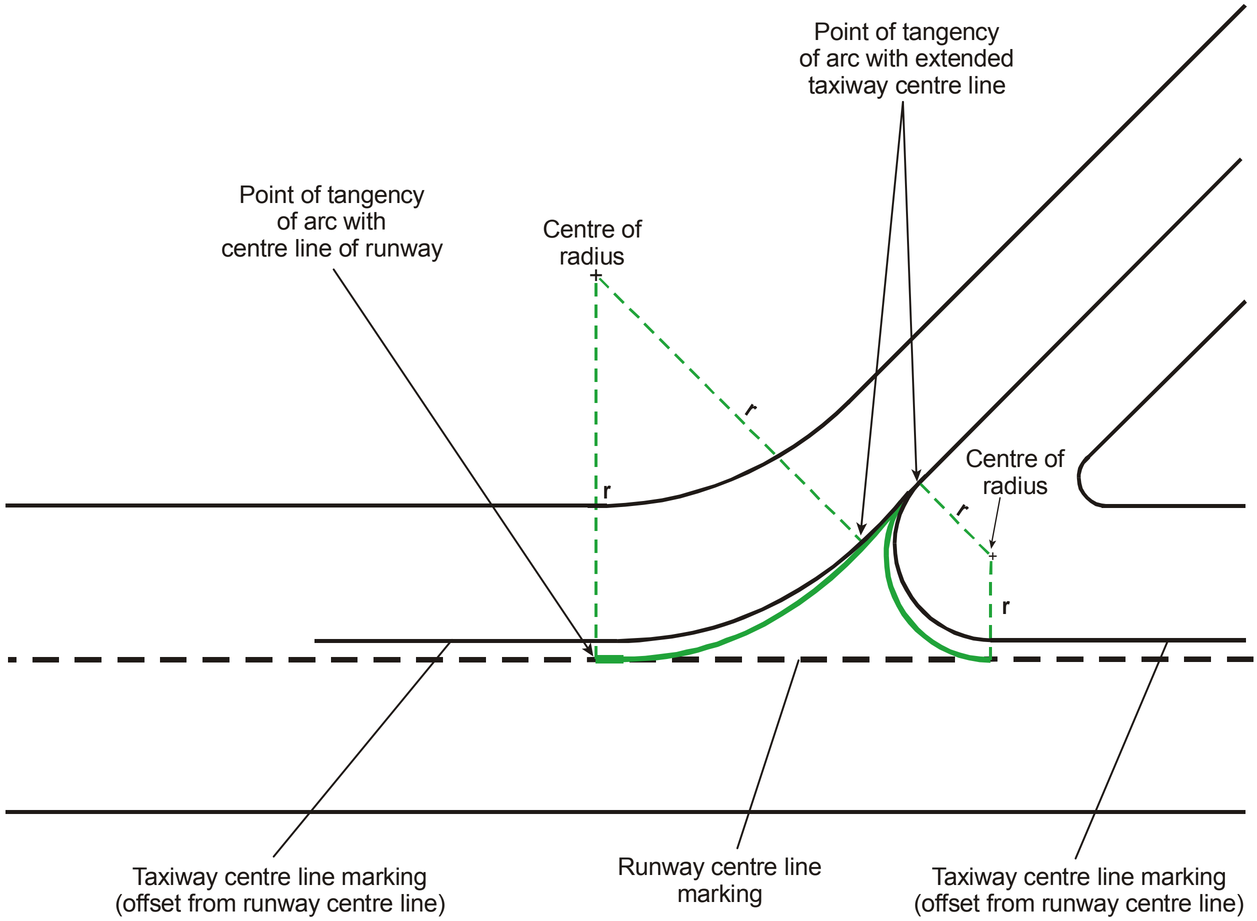

Figure 5B-15. Runway and taxiway intersections to be surveyed

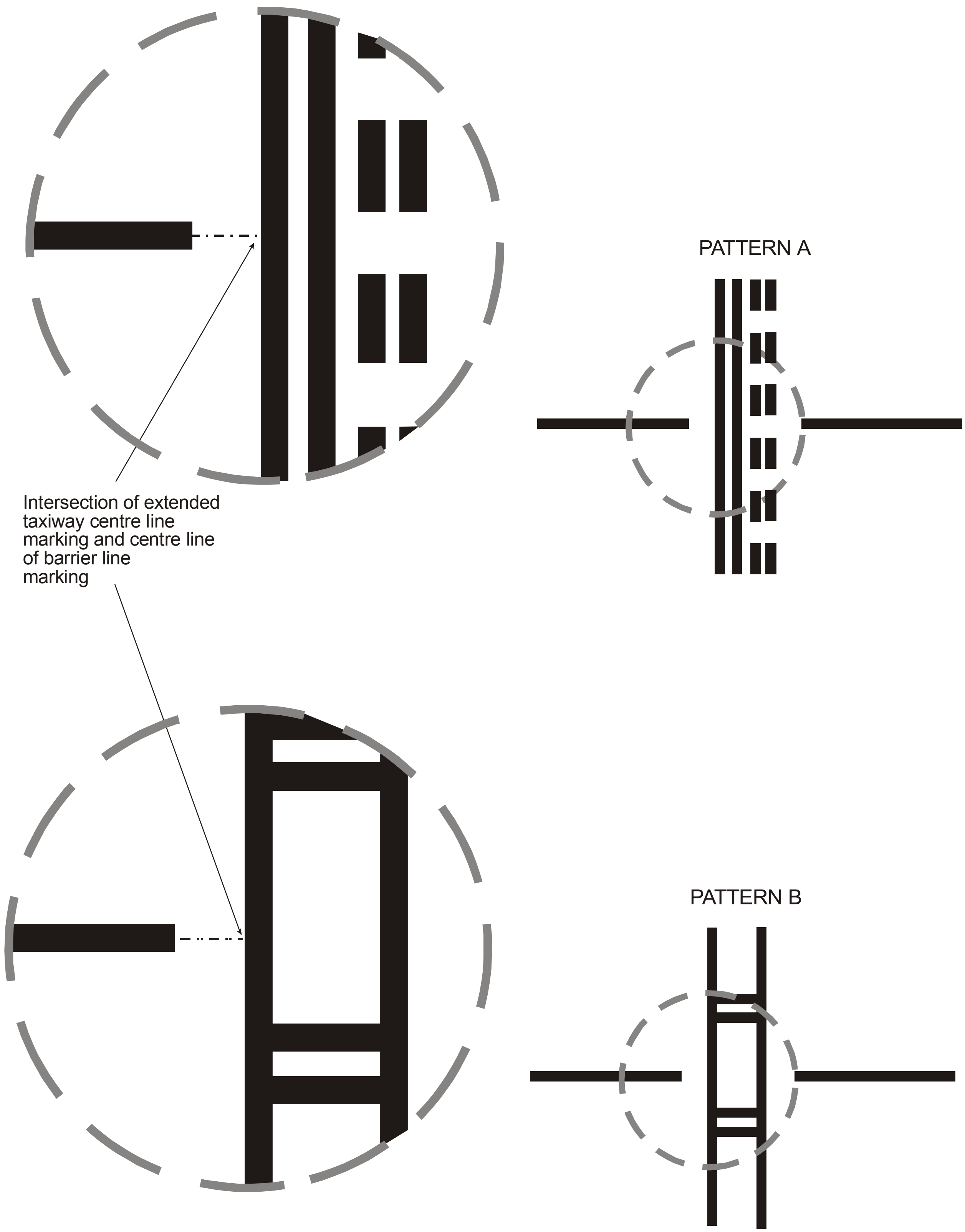

Figure 5B-16. Runway holding positions to be surveyed

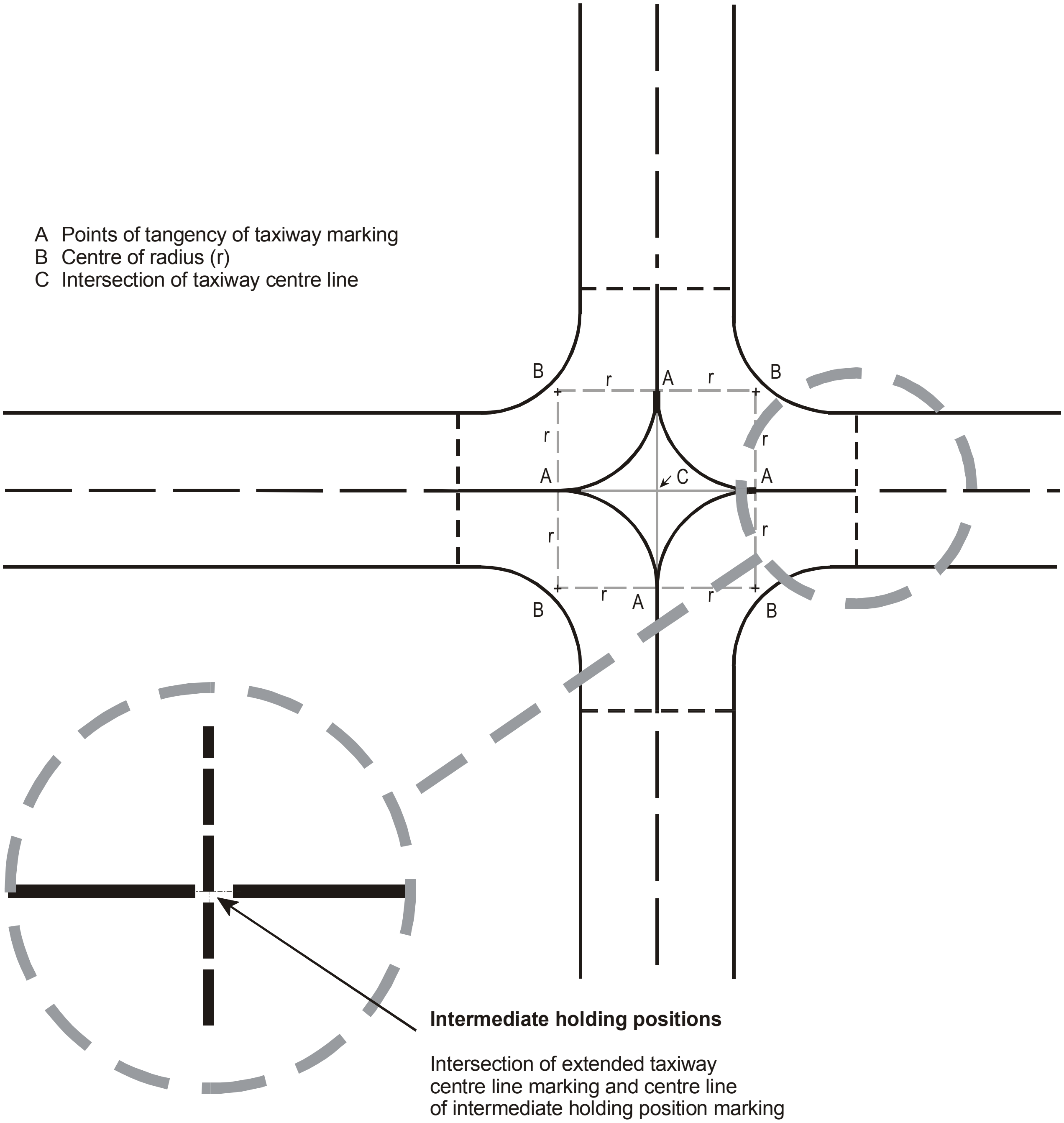

Figure 5B-17. Taxiway intersections to be surveyed

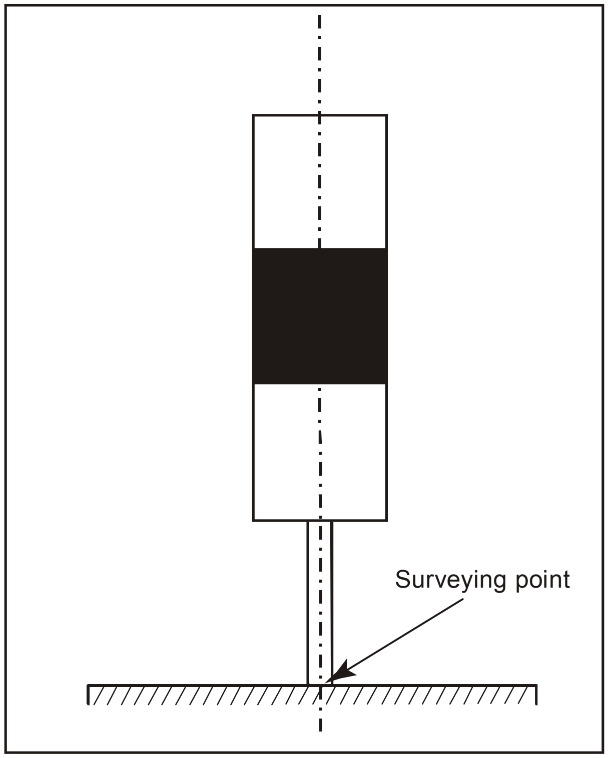

Figure 5B-18.   Helicopter air taxiway marker to be surveyed

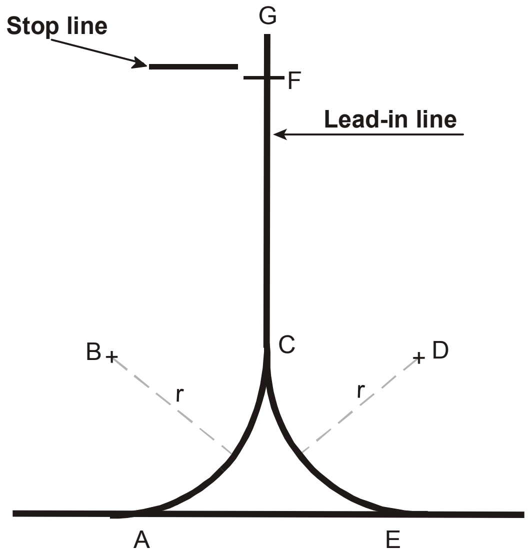

Figure 5B-19. Simple nosewheel lead-in line

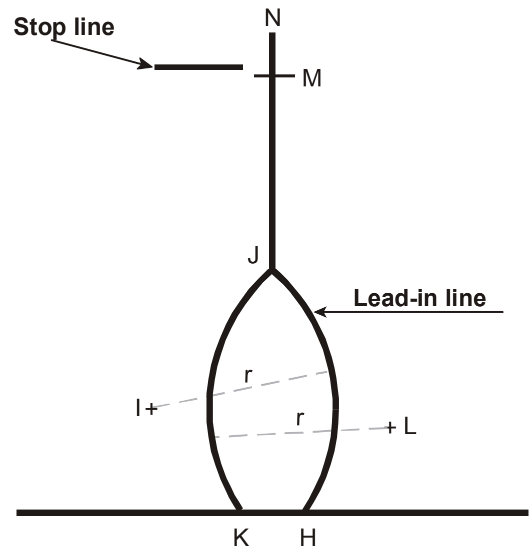

Figure 5B-20. Offset nosewheel lead-in line

Position	*Description of point to be surveyed*
A	Point of tangency of centre of lead-in marking with centre of taxilane marking
B	Centre of arc of lead-in line and radius
C	Point of tangency with centre of lead-in line marking
D	Centre of arc of lead-in line and radius
E	Point of tangency of centre of lead-in marking with centre of taxilane marking
F	Nosewheel position of parked aircraft
G	End of lead-in line marking
H	Intersection of centre of lead-in line marking and centre of taxilane marking
I	Centre of arc of lead-in line and radius
J	Centre of commencement of straight section of lead-in line
K	Intersection of centre of lead-in line marking and centre of taxilane marking
L	Centre of arc of lead-in line and radius
M	Nosewheel position of parked aircraft
N	End of lead-in line marking

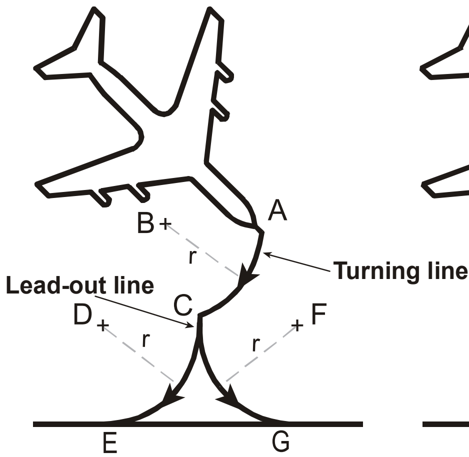

Figure 5B-21

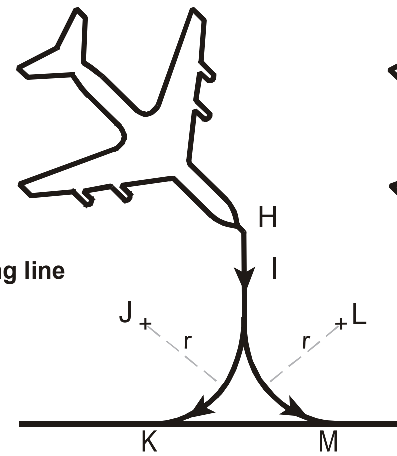

Figure 5B-22

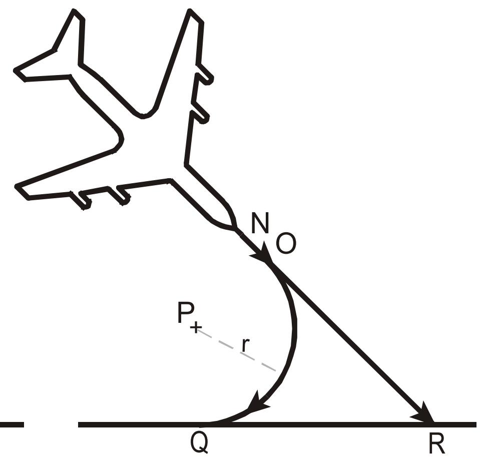

Figure 5B-23

Simple nosewheel lead-out lines

Position	*Description of point to be surveyed*
A	Centre of commencement of turning line marking
B	Centre of arc of turning line and radius
C	Centre of intersection of turning line marking and lead-out line marking
D	Centre of arc of lead-out line and radius
E	Point of tangency of centre of lead-out line marking and taxilane marking
F	Centre of arc of lead-out line and radius
G	Point of tangency of centre of lead-out line marking and taxilane marking
H	Commencement of lead-out line
I	Centre of commencement of curved section of lead-out line
J	Centre of arc of lead-out line and radius
K	Point of tangency of centre of lead-out line marking and taxilane marking
L	Centre of arc of lead-out line and radius
M	Point of tangency of centre of lead-out line marking and taxilane marking
N	Point of tangency of centre of lead-out line marking and taxilane marking
O	Centre of commencement of curved section of lead-out line
P	Centre of arc of lead-out line and radius
Q	Point of tangency of centre of lead-out line marking and taxilane marking
R	Intersection of centre of lead-out line marking and taxilane marking

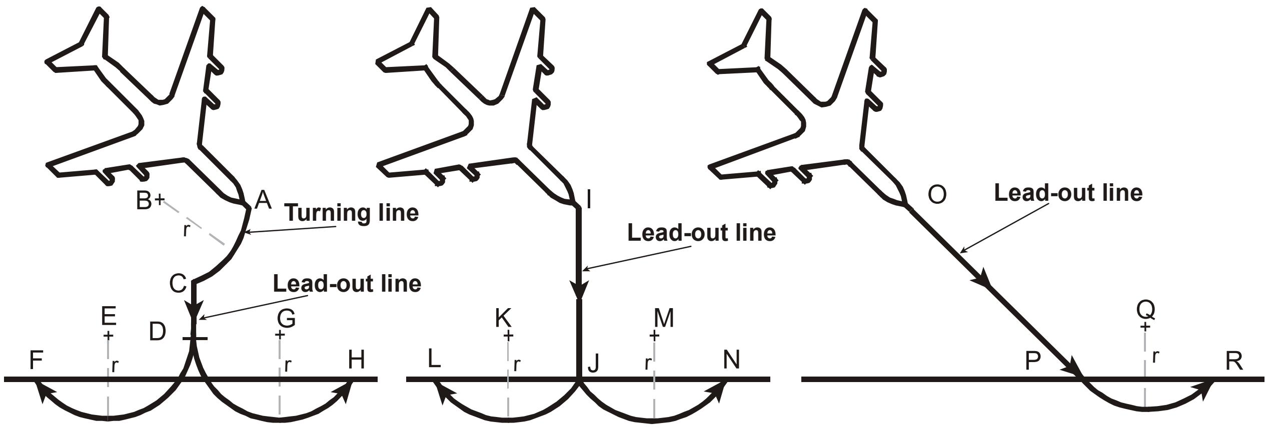

Figure 5B-24 **Figure 5B-25** **Figure 5B-26**

Offset nosewheel lead-out lines

Position	*Description of point to be surveyed*
A	Centre of commencement of turning line marking
B	Centre of arc of turning line and radius
C	Centre of intersection of turning line marking and lead-out line marking
D	Centre of end of straight section of lead-out line marking
E	Centre of arc of lead-out line and radius
F	Intersection of centre of lead-out line marking and taxilane marking
G	Centre of arc of lead-out line and radius
H	Intersection of centre of lead-out line marking and taxilane marking
I	Commencement of lead-out line
J	Centre of commencement of curved section of lead-out line
K	Centre of arc of lead-out line and radius
L	Intersection of centre of lead-out line marking and taxilane marking
M	Centre of arc of lead-out line and radius
N	Intersection of centre of lead-out line marking and taxilane marking
O	Commencement of lead-out line
P	Centre of commencement of curved section of lead-out line
Q	Centre of arc of lead-out line and radius
R	Intersection of centre of lead-out line marking and taxilane marking

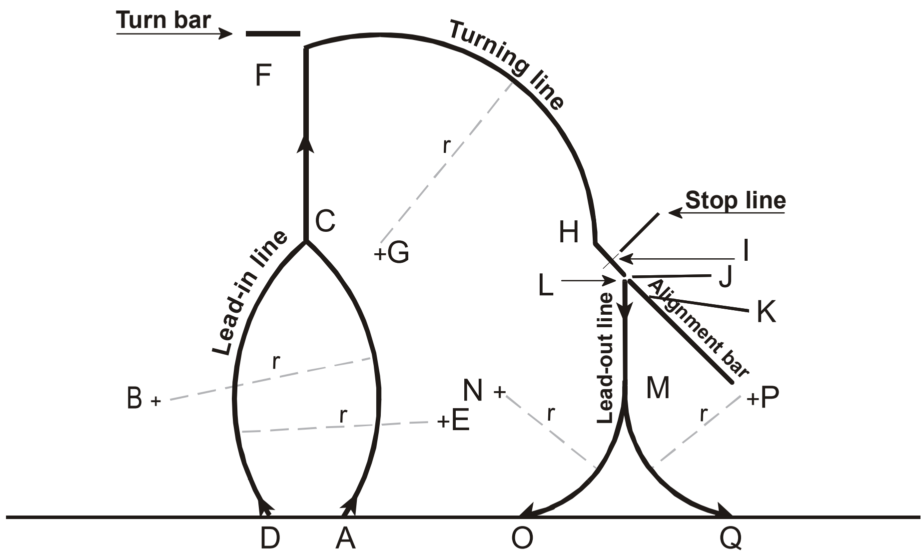

Figure 5B-27. Turning lines

Figure	*Position*	*Description of point to be surveyed*
5B-27	A	Intersection of centre of lead-in line marking and centre of taxilane marking
	B	Centre of arc of lead-in line and radius
	C	Centre of commencement of straight section of lead-in line
	D	Intersection of centre of lead-in line marking and centre of taxilane marking
	E	Centre of arc of lead-in line and radius
	F	End of straight section of lead-in line marking/commencement of turning line marking
	G	Centre of arc of turning line and radius
	H	Centre of commencement of straight section of turning line marking
	I	Nosewheel position of parked aircraft
	J	Centre of end of straight section or turning line marking
	K	True bearing of alignment bar
	L	Commencement of lead-out line
	M	Centre of commencement of curved section of lead-out line
	N	Centre of arc of lead-out line and radius
	O	Point of tangency of centre of lead-out line marking and taxilane marking
	P	Centre of arc of lead-out line and radius
	Q	Point of tangency of centre of lead-out line marking and taxilane marking

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Attachment C. SURVEY REPORTS

1. GEODETIC CONNECTION

1.1 A survey report conforming to the following general format must be provided.

Contents list — Geodetic connection

1. Receipt note signed on behalf of the commissioning organization indicating the date of receipt of the survey report, confirming its completeness and listing the distribution of copies of the report.

2. Historical data (dates and general purpose of survey, names of surveyor and survey organization, etc.).

3. Description of the method of survey.

4. Details of the datum connection and the source of the control coordinates (i.e. original descriptions and coordinate lists from the national geodetic organization or lists cross-referenced to previous surveys).

5. Control network diagram.

6. Survey station descriptions.

7. Schedule of points surveyed showing date of monumentation, description and survey.

8. Quality control report indicating equipment calibration information and the method of checking of the survey. Demonstrable evidence that the accuracy requirements have been met.

1.2 Records of actual observations must be provided in a separate indexed volume. Cross-references to observations must be made in the survey report.

2. AERODROME/HELIPORT SURVEY

2.1 A survey report conforming to the following general format must be provided.

Contents list — Aerodrome/heliport survey

1. Receipt note signed on behalf of the commissioning authority indicating the date of receipt of the survey and listing the distribution of copies of the report.

2. Historical data (dates and general purpose of survey, names of surveyor and survey organization, etc.).

3. Description of the method of survey.

4. Details of the observations made, cross-referenced to the control survey.

5. Navigation elements survey plan and cross-referenced witness diagrams (where necessary).

6. Schedule of points surveyed showing coordinates and date of survey, including diagrams as required.

7. Quality control report indicating equipment calibration information and the method of checking of the survey. Demonstrable evidence that the accuracy requirements have been met.

2.2 Records of actual observations must be provided in a separate indexed volume. Cross-references to observations must be made in the survey report.

3. EN-ROUTE SURVEY

3.1 A survey report conforming to the following general format must be provided.

Contents list — En-route survey

1. Receipt note signed on behalf of the commissioning authority indicating the date of receipt of the survey and listing the distribution of copies of the report.

2. Historical data (dates and general purpose of survey, names of surveyor and survey organization, etc.).

3. Description of the method of survey.

4. Details of the coordination of individual navigation aids.

5. Survey diagram showing the local survey connection by which the coordinates of the centre of the aid were obtained.

6. Schedule of points surveyed showing coordinates and date of survey.

7. Quality control report indicating equipment calibration information and the method of checking of the survey. Demonstrable evidence that the accuracy requirements have been met.

3.2 Records of actual observations must be provided in a separate indexed volume. Cross-references to observations must be made in the survey report.

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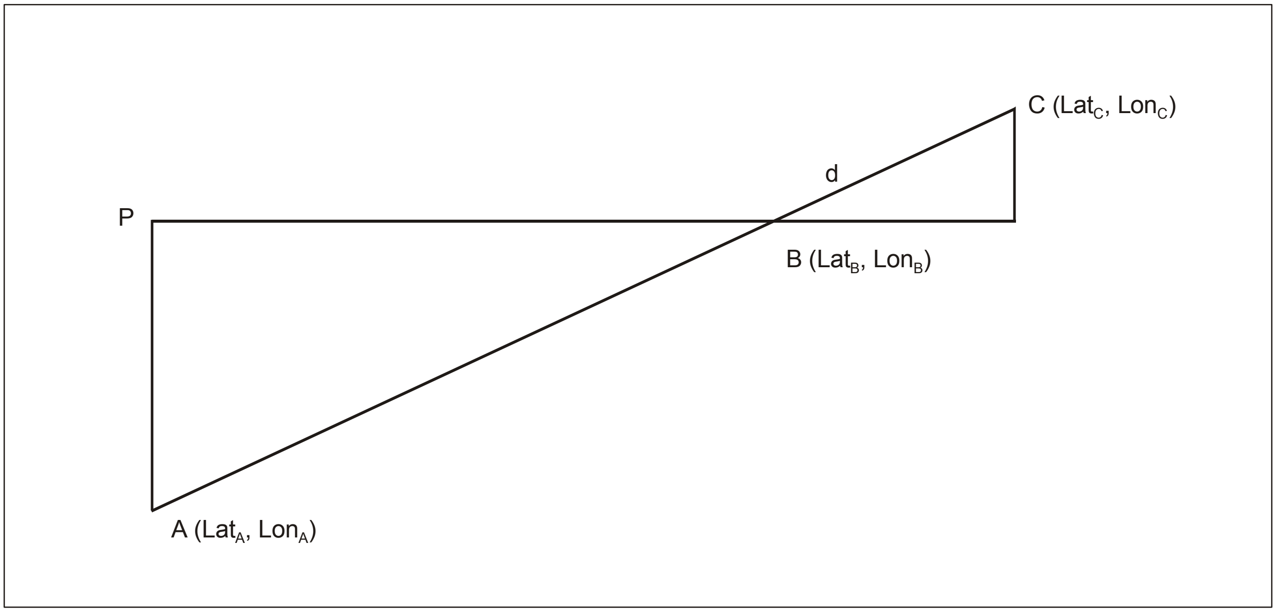
Attachment D. COMPUTATION OF THRESHOLD COORDINATES

Where the centre line point actually surveyed does not coincide with the threshold, the threshold coordinates may be derived by computation of coordinates of a threshold longitudinally offset from the point surveyed by using the following method:



(Angles in decimal degrees.)

Given: A (LatA, LonA) — runway centre line point
B (Lat_B, Lon_B) — surveyed threshold point
d (metres) — longitudinal offset to true threshold

Find: C (Lat_C, Lon_C)

$$PB = (Lon_B - Lon_A) \times 1\,852 \times 60 \times \cos[(Lat_B + Lat_A)/2]$$
$$PA = (Lat_B - Lat_A) \times 1\,852 \times 60$$
$$AB = +\sqrt{(PB^2 + PA^2)}$$
$$k = d/AB$$

$$Lat_C = Lat_B + k(Lat_B - Lat_A)$$
$$Lon_C = Lon_B + k(Lon_B - Lon_A)$$

Note 1.— Using the naming convention described, the above formula works for all cases. The accuracy of the formula may be improved by using a more accurate local figure than 1 852 m for 1 NM.

Note 2.— Longitudes west of Greenwich may be entered as negative. Where the offset is from B towards A, the dimension d may be entered as negative.

Note 3.— These are approximate formulae and may only be used where d is small (i.e. less than 1 km and less than AB).

Chapter 6

QUALITY ASSURANCE

6.1 QUALITY DEFINITIONS

6.1.1 It is essential to have a common understanding of the terminology used in discussing quality issues. Most people will have different interpretations of the meaning of "quality" based on their personal experience. The following definitions are included to establish consistency.

Procedures. The method used, i.e.: how the responsibilities for the task should be assigned; what should be achieved in the tasks; and what should be recorded as the associated quality record.

6.1.2 A procedure is not equipment-specific. It deals with what should be achieved having satisfied the steps of the procedure. Work instructions are the detailed "how to" descriptions, for example, how to operate a particular instrument or piece of equipment.

Quality. Totality of characteristics of an entity that bear on its ability to satisfy stated and implied needs (ISO 8402*).

Note.— Entity is an item that can be individually described and considered (ISO 8402).*

6.1.3 In this case, it indicates the ability of a product to **consistently** meet its stated requirements, i.e. it is qualified for its specified purpose. There is no single or absolute measure of quality although statements about the quality of a process or item may be based upon physical measurements and observations, for example, quality level.

Quality assurance (QA). All the planned and systematic activities implemented within the quality system, and demonstrated as needed, to provide adequate confidence that an entity will fulfil requirements for quality (ISO 8402*).

6.1.4 In other words, QA is the process of ensuring that stated quality specifications are incorporated in the final product, by use of pre-defined methods. Once a method has been proven to produce the required product successfully, a system is required that can assure that the method or methods are followed correctly each time the process is repeated. All activities and functions that affect the level of quality of a product are of concern to QA which is achieved through the use of a quality system.

Quality control. The operational techniques and activities that are used to fulfil requirements for quality (ISO 8402*).

Quality level. The extent to which the customer's needs have been met. A quality level of 100% means that there has been a complete conformance to specification **every** time.

Quality management. All activities of the overall management function that determine the quality policy, objectives and responsibilities, and implementing them by means such as quality planning, quality control, quality assurance and quality improvement within the quality system (ISO 8402*).

6.1.5 This could be interpreted as the implementation of QA.

Quality record. Documented evidence of tasks carried out which demonstrates that the required results have been achieved and provides sufficient links to other quality records to ensure traceability.

Quality specifications. The minimum, pre-defined specifications that must be met to fulfill the stated quality requirements. A quality system provides the management control to assure the required quality specifications are achieved.

Quality system. The organizational structure, procedures, processes and resources needed to implement quality management (ISO 8402*).

* ISO Standard 8402 — *Quality Management and Quality Assurance —Vocabulary*, Second Edition

Requirements for quality. Expression of the needs or their translation into a set of quantitatively or qualitatively stated requirements for the characteristics of an entity to enable its realization and examination (ISO 8402*).

Resolution. A number of units or digits to which a measured or calculated value is expressed and used.

Traceability. Ability to trace the history, application or location of an entity by means of recorded identifications (ISO 8402*).

Validation. Confirmation by examination and provision of objective evidence that the particular requirements for a specific intended use are fulfilled (ISO 8402*).

Verification. Confirmation by examination and provision of objective evidence that specified requirements have been fulfilled (ISO 8402*).

Note.— Objective evidence is information which can be proved true, based on facts obtained through observation, measurement, test or other means (ISO 8402).*

Work instructions. Actual steps to carry out a procedure. These are the details that are specific, for example, to a particular piece of equipment used in the production process.

6.2 QUALITY ASSURANCE (QA)

6.2.1 Need for a quality assurance system

The objective of WGS-84 implementation is to produce coordinate data referenced to a common datum in which a high degree of confidence can be placed in the accuracy and integrity of the data. The method used to acquire all the positional data to the required standard has to address the problems of the size of the task. All the coordinates produced should be traceable back to their origin along an unbroken trail. Each point is unique so that quality control checks on a sample would not be suitable. Considering the large number of points and the geographical spread, it would be impractical to independently check every single point. However, it is possible to check the method by which the data are acquired. This can be achieved by QA.

6.2.2 Quality assurance and WGS-84 implementation

6.2.2.1 QA is about preventing errors, rather than fixing them. To do this, it is essential to understand the requirement. The various tasks involved need to be identified and managed efficiently. This is done by using a quality system or quality management system (QMS). The basic elements of such a system could be (see Figure 6-1):

a) *Organization* (the management structure). It is very important that the responsibilities for the operation are stated and understood by all concerned, i.e. everyone knows who does what.

b) *Planning/procedures.* Identifying the tasks to be done and developing procedures necessary for the production process.

c) *Documentation.* Procedures should be written down to enable consistency of application by different personnel. Documents can be updated, but under an authorized control procedure. Written quality records are needed to provide traceability if there is a problem to be located.

d) *Assessment.* The most important part of any quality system is the method of assessment used, i.e. the audit process. It provides the checks that show whether the procedures are being used correctly and whether they are achieving the required results. It initiates the loop to make improvements where necessary. The aim of an assessment is to provide constructive recommendations for improvement where there are non-conformances and to establish confidence in the methods where there is conformance.

e) *Review.* The process of considering the assessment result and implementing any necessary changes through a corrective action procedure.

Note.— The International Organization for Standardization (ISO) 9000 series of quality assurance standards provides a basic framework for the development of a quality assurance programme. The details of a successful programme are to be formulated by each State and in most cases are unique to each State's organization.

6.2.2.2 Figure 6-2 is an example of a QMS structure applicable to the QA of the acquisition of new data. It illustrates:

a) the objectives and responsibilities as set up within the national administration;

b) various planning tasks, including:

   i) decision to use existing data or to resurvey;

   ii) type of survey: geodetic, aerodrome or en-route;

iii) accuracy requirements; and

iv) briefing of survey contractors on the accuracy requirements and safety issues, and evaluating their suitability before awarding contracts;

c) two outcomes from the assessment process (audit):

i) conformance, in which case, the data can then be processed; and

ii) non-conformance, where the process flow should then loop back to the planning stage via a corrective (remedial) action procedure.

6.2.3 Safety aspects in surveying

6.2.3.1 A quality system must take into account safety aspects in surveying. Involvement in safety has various legal positions to be considered and will vary within each administration. A quality system, however, needs to encourage safe working practices as any accident is a reflection on the quality of the service, for example, making sure that the survey teams are well briefed on the type of survey they have to do and are aware of any local procedures. This is particularly important where work at aerodromes is concerned.

6.2.3.2 Prior to awarding any contracts, evaluation of the survey company's experience should be considered, i.e. how many aerodrome surveys they have conducted. To follow the requirements of a QMS, quality records should be kept, such as a checklist of the briefing of the survey team. If there is no acknowledgment that the survey team has been briefed on the aerodrome safety requirements, then the team should not proceed.

6.2.4 Quality plans

The QMS described above has been designed purely for the QA of the origin of WGS-84 data, largely relying on the acquisition by survey teams. The scope of this system does not include the management of all the navigation data processes that may be the responsibility of an AIS department. It is possible, however, for this QMS to function as a subsidiary element within a total quality system having a wider scope. The subsidiary QMS is referred to as a quality plan. For example, the QMS described in Figure 6-2 has been labelled the State quality plan. This plan can be incorporated within an existing administration's quality system or as part of a new one along with other quality plans, such as one for managing the onward flow of data via database storage and publication. Figure 6-3 gives an example of such a total data management system.

6.2.5 Information needed to define data quality

If integrity is to be assured and demonstrable, all coordinates must be traceable to their source by an unbroken trail. While the Cyclic Redundancy Check (CRC) will be used by receivers of data to confirm correct receipt, it is not sufficient to define the quality of the data. Therefore, a record must be kept of all changes made. The following are some of the elements of a sample set of the required data record:

a) data accuracy;

b) data origin;

c) details of changes made to the data;

d) reason for the data change;

e) references associated with the data change;

f) the source of the data change;

g) the identity of the person making the change;

h) date of the change.

6.2.6 Procedures to ensure traceability

6.2.6.1 The quality records must be kept by the organization carrying out the data modification. While an indicator allowing the retrieval of this information will need to be associated to data transferred to the next intended user, together with the CRC, the quality record itself does not need to be sent.

6.2.6.2 The quality records may be in electronic or hard copy format. To provide for the unbroken trail, certain change information must remain with the data item throughout the data manipulation and use and must, wherever possible, be stored in an associated field or record.

6.2.7 Procedures to ensure integrity

6.2.7.1 The accuracy of data (see Chapter 2) is determined at the point where the data originate. In the case of surveyed data, procedures necessary to ensure the established accuracy can be found in Chapter 5 — Surveying Guidance. Procedures for calculating points must not only take into account the accuracy of the known (surveyed) data, but must also ensure that subsequent mathematical manipulation maintains the accuracy requirements set by the data model. Declared points must be declared to the accuracy required by the data model provided in Chapter 2.

6.2.7.2 If data integrity is to be assured, there must be clearly defined procedures for all stages of the navigation data process, from the point where the data are originated to the point where the data are used. Apart from rigorous manual independent verification, there is little that can be done to ensure the integrity of data held in a manual system. However, once the data are held on electronic media, there are a number of options available. When choosing appropriate methods to protect the integrity of electronically stored data, consideration must be given to the integrity requirements for the data and the risk posed to that data.

6.2.8 Manual data entry

The transfer of data from written or printed form into an electronic format is the greatest potential source of error in the entire process. Careful consideration must be given to the means by which this transfer is to be performed and verified if end-to-end data integrity at the required levels is to be achieved.

6.2.9 Data validation checks

Data validation checks, which can be performed once the data item is held in electronic format, will detect many of the errors induced by manual data entry, but it is doubtful whether integrity can be improved even by one order of magnitude on the basis of validation checks alone.

6.2.10 Software aspects

6.2.10.1 Whenever data are manipulated by a computer programme, even if it is simply to extract an item from the database and output it onto magnetic media, there is a risk that, as a result of software error, the resultant data item will not be a true copy of the original. Accordingly, all software used to manipulate data must be subject to rigorous testing, verification and validation.

6.2.10.2 In addition to the threat to data integrity posed by a software fault, there is a threat from computer viruses which may be introduced via an executable code in applications software and utilities. This aspect must also be addressed by the configuration management system.

6.2.11 Data retention aspects

Although hardware reliability has improved markedly over the years, there is still a risk of corruption from component failure or power surge/spike. Detection of corruption caused by hardware faults can be improved by the use of validation and verification checks at regular intervals.

6.2.12 Data transfer aspects

The risk to data, while being written to or read from magnetic/optical storage media, depends on the devices used and the methods employed in the packing and handling of the media. Protection is provided by the software controlling the reading from/writing to the media. To achieve the protection of data while stored or being transferred, the CRC must be used. (See Chapter 7 for details.)

FIGURES FOR CHAPTER 6

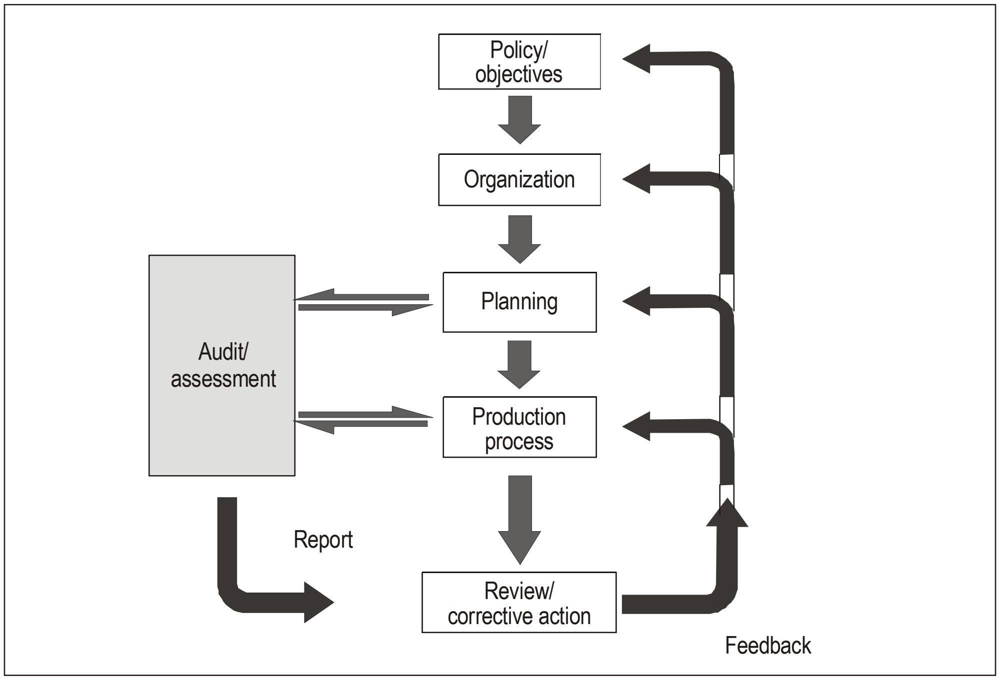


Figure 6-1. Basic structure of a quality system (QA loop)

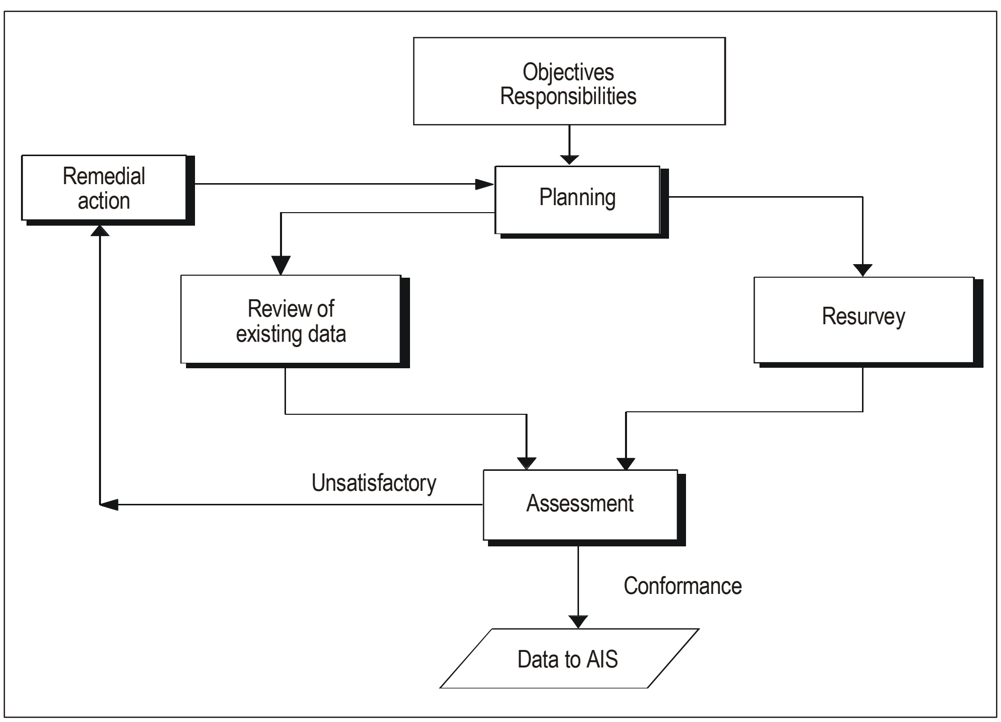


Figure 6-2. Model of a State quality plan

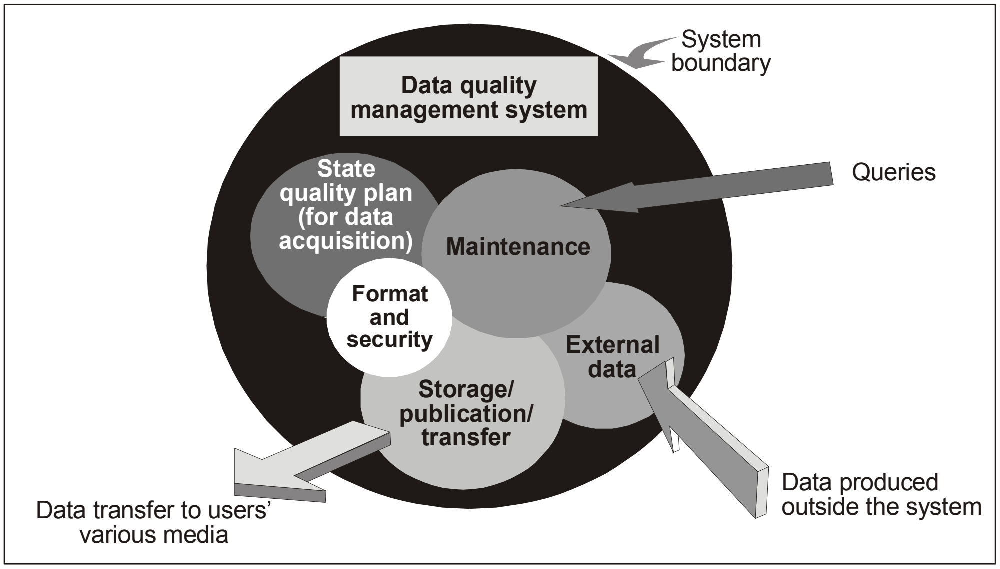


Figure 6-3. Quality plans within a quality system

Chapter 7

DELIVERABLES

7.1 SURVEY REPORTING REQUIREMENTS

7.1.1 It is important to know what type of data has been surveyed and to what level of accuracy and resolution this was done. The classification of data types, accuracy and resolution was described in Chapter 2. In addition to the production of accurate positional data, the other aim of WGS-84 implementation is to be able to recall the history of that data. It is the survey report that will provide the details of the acquisition method used. This is the quality record that will be used for assessment and future reference.

7.1.2 It is therefore essential that the surveyors record all the necessary information. In this way, all survey work undertaken to determine the coordinates of navigation facilities can be reported in a predefined format. Where an existing national reporting practice differs from that shown in this manual, national administrations may make a case in support of the national practice, where this can be shown to be compatible.

7.1.3 An advantage to having consistency in the report through the use of standardized formats could be realized in the assessment (audit) phase. Checklists would facilitate an efficient audit process particularly where large numbers of surveys are involved. Further advantages of checklists are their use in briefing the survey teams on the requirements of the surveying standards and as a guidance document for the reporting requirement, the benefit being consistency for the administration.

7.2 BASIC REPORTING STRUCTURE

7.2.1 The following is an example of the basic reporting format structure:

a) *Three types of reporting formats.*

   i) Geodetic survey,

   ii) En-route survey,

   iii) Aerodrome/heliport survey.

There is also the case of coordinate data that have been produced by conversion from an existing data set, which itself should have originated from one of the above survey types.

b) *Common reporting elements.* A number of topics are common to each of the report formats.

   i) Historical data should describe the general survey information as follows:

   — the purpose,

   — the date, and

   — the surveyors' names and the company.

   ii) Survey method used: This is the actual way the survey was carried out, not just a description of the theory behind the technique used.

   iii) Diagrams: Where relevant, diagrams should be included, i.e. for station descriptions, control networks and threshold descriptions.

   iv) The quality control report should provide information on the equipment calibrations carried out. It should also describe the methods used to check the survey and, in particular, show evidence that the required accuracy for the particular data type has been achieved.

   v) Observations: Records of the actual observations should be provided in a separate volume. Cross-references should be made to the survey report.

c) *Aerodrome/heliport survey report format.* The following lists a complete reporting format for an aerodrome/heliport survey.

1. A receipt note signed on behalf of the commissioning authority, indicating the date of receipt of the survey and the number of copies of the report.

2. Historical data giving the dates and purpose of the survey, the survey company names and personnel.

3. Description of the method of the survey.

4. Details of the observations made with cross-references to the control survey.

5. A facility survey plan with cross-referenced witness diagrams (where necessary).

6. Schedule of the points surveyed giving the coordinates and the date when surveyed.

7. QC report giving equipment calibration details and describing the methods used to check the survey. In addition, evidence should be provided to show that the accuracy requirements have been met.

8. Actual observations (provided in a separate volume), indexed so that cross-references can be included in the report.

7.3 FORMATS, STANDARD ALGORITHMS AND WORKING PRACTICES

The following data format, standard algorithms and working practices are suggested for common use.

7.3.1 Universal Data Delivery Format (UDDF)

7.3.1.1 The Universal Data Delivery Format (UDDF), submitted by the United States (Federal Aviation Administration document No. 405, *Standards for Aeronautical Data and Related Products*), is a digital delivery system which provides aeronautical and other data, including aerodrome/heliport, runway, navigation aid and obstacle, in a standard ASCII format. This information can be easily read into the user data files and databases. It details the format to be used when reporting surveyed data to the AIS.

7.3.1.2 Inquiries, comments or recommendations concerning UDDF are encouraged and should be addressed to:

Robert Fisher
Manager, FAA Programs
Photogrammetry Branch, NGSD
1315 East-West Highway
Silver Spring, Maryland
20910

7.3.1.3 FAA document No. 405 is organized as follows:

a) *Structure overview* describes the general structure of the UDDF format.

b) *Conventions* describes UDDF conventions, such as the use of record type separators and field delimiters.

c) *Annotated file listing* presents annotated UDDF files which include code numbers at the beginning of each field. The code numbers have been decoded indicating data description, field type and field columns.

7.3.1.3.1 *Structure overview.* The UDDF is implemented in seven files, each file providing data for one of seven survey types, as follows:

a) airport obstruction chart (AOC);

b) area navigation approach precision conventional landing (APC)

c) area navigation approach non-precision conventional landing (ANC);

d) area navigation approach precision vertical landing (APV);

e) area navigation approach non-precision vertical landing (ANV);

f) engine out departure (EOD);

g) special purpose (SPL).

7.3.1.3.1.1 Each of these files is organized into the following data sections.

a) Airport data furnishes airport name, ICAO location indicator, FAA site number, survey date, survey edition, magnetic declination and other airport and survey information.

b) Runway data furnishes thresholds and displaced thresholds, stopway, blast pad and other runway-related information.

c) Navigation aid data furnishes navigation aid information including computed distances between navaids and selected runway points.

d) Obstacle data furnishes information on obstacles, including computed distances from and heights above selected runway points.

e) Special notices lists miscellaneous information that cannot be logically included with the other data, such as advisories of possible surface penetrations by vessels.

7.3.1.3.2 *Conventions.* UDDF information is furnished as ASCII files. The file name identifies the airport, approximate survey date and survey type. Each field is delimited by the vertical broken line symbol and data sections are separated within the file by the "@" symbol. Data sections within a section, such as data for individual runways within the runway section, are separated by the "#" symbol. The end of the file is indicated by "EOF".

7.3.1.3.3 *Annotated file listing.* An annotated file with field descriptions is required (see Figure 7-1).

7.3.1.3.4 When entering data into the database, it will be necessary to:

a) confirm the data against the original survey data;

b) enter the data under strict quality control procedures; and

c) verify entered data.

7.3.2 Cyclic redundancy check (CRC) algorithm

7.3.2.1 For those civil aviation authorities who will be developing the electronic aeronautical database, it is specified that the transfer of data, whether from personal computer to personal computer, within a database, or digitally over communication networks, be monitored by a cyclic redundancy check (CRC) determined for the batch of data to be transferred. A CRC can also be used to monitor the integrity of specific records within the database. Data integrity values to be maintained for specific data within a database or while being transferred are grouped in Tables 2-1 to 2-5.

7.3.2.2 CRC is an error detection algorithm capable of detecting small changes in a block of data. For those civil aviation authorities who will continue to develop and transfer data using manual processes, a CRC value could be attached to the data set so that each succeeding user who receives the data could compare the original CRC to a recalculated value. A hard copy Aeronautical Information Publication (AIP) could be created using a desktop publishing system that would provide the aeronautical data in an electronic format. Individual data or all the data contained in one page could then be "wrapped" into the CRC to ensure the protection of data. The resulting calculated value of the CRC applied should be printed either together with the specific individual data or for the whole page. In this way, the CRC value attached to the original data values could be monitored from data creation through publication. Recipients of data, such as the data preparation agency and the avionics manufacturer, would then be able to verify the integrity of the values received from the civil aviation authority prior to inputting into the database by comparing the printed CRC value with the value obtained from their own, independent CRC check.

7.3.2.3 CRC is a mathematical process whereby a sequence of N data bits is manipulated by an algorithm to produce a block of n-bits, known as the CRC, where n is less than N. A check of the integrity of the data can be performed by comparing the result of the application of the CRC algorithm with the declared expected result. A failure of the mathematical routine to regenerate the CRC value from the data indicates that either the data or the CRC has become corrupt and the data are, therefore, no longer reliable. By careful choice of the algorithm employed, in conjunction with the relative values of n and N, it can be guaranteed that more than a specified proportion of the original N-bits must be corrupted before there is any possibility of the same resultant n-bit code being produced. For a given CRC formula, the ratio between n and N will determine the integrity level achieved for any data element.

7.3.2.4 In summary, the characteristics of a CRC are:

a) it is better than check sums or parity bits for error detection;

b) the elements of generating a CRC are

— a data block divided by a generating polynomial,

— the CRC is resulting remainder,

— the CRC is tagged onto the end of the data block, and

— no errors gives the remainder from division as zero;

c) the assurance of error detection (see relationship between data integrity and CRC length as contained in Table 7-1).

7.3.2.5 *Standard CRC algorithms*

7.3.2.5.1 The generator polynomial of a CRC algorithm is measured in bit size where the polynomial coefficients are binary values equal to 0 or 1. The level of integrity protection provided by a specific generator polynomial is a function of the highest order term in the polynomial. The higher the term, the higher the level of protection. From the following generator polynomials:

a) $1 + x^5 + x^{12} + x^{16}$ is the CRC-CCITT algorithm; and

b) $1 + x + x^3 + x^5 + x^7 + x^8 + x^{14} + x^{16} + x^{22} + x^{24} + x^{31} + x^{32}$ is the CRC-32Q algorithm.

7.3.2.5.2 The CRC-CCITT algorithm is a 16-bit algorithm and provides less protection than the CRC-32Q algorithm, which is a 32-bit algorithm. The above CRC algorithms are suggested for uniform application by States worldwide.

7.3.2.6 *Using a CRC for checking the correctness of data*

7.3.2.6.1 Integrity cannot be added once it is lost. Thus, the integrity assurance must be provided from origination to final application. It is therefore suggested that when a data item is entered into an electronic format, it will need to be provided with the CRC, thereby providing the capability for verification on its transfer to the next intended user. Then, and at each subsequent step, the receiver must confirm the data validity to provide assurance that data have not been corrupted while stored or during transmission.

7.3.2.6.2 The insertion of data into the database at the start of the process is therefore a most critical point in the whole process. At this time it will be necessary to:

a) confirm the correctness against the original survey data;

b) enter data under strict quality control procedures; and

c) execute checks to verify data following entry.

7.3.2.6.3 For the CRC to be used for checking the correctness of data, the chosen CRC must meet the level of integrity required for the individual data items to be transferred. This CRC value will need to be associated with the smallest related set of data. Thus, for much of the WGS-84-related subject of implementation, this smallest element will be a single point. A suitable set of data for a CRC check could also be a procedure and, ultimately, a whole aeronautical database.

7.3.2.7 *Different computer formats and their effect on CRC*

Ideally, once formed, the CRC would remain unchanged until its application in the flight management system (FMS) or RNAV system. However, data formats change during the process and a CRC is only valid for a particular data format. For example, data are held in a computer in a format dependent upon word length which is unlikely to be the same in the database of the supplier or the RNAV/FMS. Thus, it will be necessary to recalculate the CRC at various steps in the management of data. This recalculation must be carried out under strict quality control if the CRC is to remain a valid indication of integrity.

7.3.2.8 *Integrity and CRC length*

7.3.2.8.1 CRC offers absolute assurance of error detection when there is only a single period of "burst error" within the stream of data that was subjected to the CRC, provided that the substring containing all affected bits is shorter than the length of the CRC employed. If the separation of corrupted bits is greater than the length of the CRC, the probability of an undetected error is the probability that the same CRC will be produced from both

Table 7-1. Integrity and CRC length

	Lengths of CRC	
Integrity	*Bits*	*Characters*
3.9×10^{-3}	8	1
1.5×10^{-5}	16	2
6.0×10^{-8}	24	3
2.3×10^{-10}	32	4

the original and the corrupted data. Assuming an "even" mapping of valid data strings to the CRC, the probability of an undetected error rises to a maximum of 2-n when the number of bits protected by the CRC is several times the length of the CRC itself.

7.3.2.8.2 Table 7-1 gives the length of the CRC required to achieve different levels of assurance of detection of multiple bit error in the data, where there is no guarantee that the separation of the bits in error is less than the length of the CRC. In order to achieve an integrity assurance of 1×10^{-8}, it is therefore necessary to employ a 32-bit CRC.

7.3.2.9 *Example of CRC generation*

7.3.2.9.1 The algorithm by which a CRC is produced is defined by a generating polynomial (GP). The GP for an n-bit CRC is of order n; the coefficients are either 0 or 1, with the constraints that the polynomial is primitive and that the coefficients of x^0 and of x^n are both 1. To generate a CRC, a data block is divided by a GP. The resulting remainder, the CRC, is usually tagged onto the end of the data block. When the data are subsequently checked, an identical division is performed on the data although the remainder is now included. If no errors have occurred, the remainder from this division should be 0.

7.3.2.9.2 This can be shown with the following mathematics, carried out using modulus 2 arithmetic.

Let:

D = data, GP = generator polynomial, Q = quotient and R = remainder.

(Suffixes O and R denote Transmitter and Receiver, respectively.)

Then:

D = QO × GP + RO (at origination)

D + RO = QR × GP + RR (at receiver)

which can be represented as D = QR × GP + RR + RO.

It follows that:

QO × GP × RO = QR × GP + RR + RO.

Cancel GP and RO to give:

QO = QR if RR = 0 (i.e. no errors).

7.3.2.9.3 A CRC can be implemented with simple shift registers and exclusive OR (XOR) gates. For clarity, the following example is performed manually.

7.3.2.9.4 *CRC generation.*

7.3.2.9.4.1 An example of the generation of a CRC is shown at Figure 7-2. The data is 11011001, the GP is 11001 and the CRC is 4 bits long.

7.3.2.9.4.2 One of the prerequisites for the GP is that it is 1 bit longer than the CRC.

7.3.2.9.4.3 Four zeros are appended to the data (the length of the CRC) and the data (with the four zeros added) are XORed with the GP. This operation yields a quotient and a remainder (the CRC).

7.3.2.9.5 *Validation.* The CRC is now appended to the data. When the receiver of the data passes this string through the same calculation again, there will be a zero remainder if no errors have occurred (see Figure 7-3).

FIGURES FOR CHAPTER 7

```
0          1           2           3           4           5           6           7
1234567890123456789012345678901234567890123456789012345678901234567890123456789012345678901...
|ASR (MFR)     |    422306.6000|    -1225146.7000|   1310.0|         |             |         |
 60                61           62                  63        64         65            66

|DME (14)      |    422140.0470|    -1225201.8010|   1334.0|         |             |         |
|GS (14)       |    422242.4910|    -1225224.7530|   1297.1|         |             |         |
|GS (14) PP    |    422241.0590|    -1225229.7230|   1299.7|         |          400|     1081|
|LOC (14)      |    422140.1380|    -1225157.8070|   1318.9|         |          998|         |
|LMM (14)      |    422321.0000|    -1225250.6000|         |         |         3250|         |
|LOM (14)      |    422703.2000|    -1225448.2000|         |         |        27420|         |
|VORTAC (OED)  |    422846.5000|    -1225446.7000|   2080.0|         |             |         |
```

Items 1 to 6 are UDDF field descriptions with the following meaning:

1. AIRPORT IDENTIFIER/A6/2-7
2. AIRPORT SITE NUMBER/A10/9-18
3. FAA REGION/A4/20-23
4. SURVEY TYPE/A6/25-30

   AOC — AIRPORT OBSTRUCTION CHART SURVEY
   EOD — ENGINE OUT DEPARTURE SURVEY
   NPA — NON-PRECISION AREA NAVIGATION APPROACH SURVEY
   PAA — PRECISION AREA NAVIGATION APPROACH SURVEY
   SPL — SPECIAL PURPOSE

5. OC NUMBER/I5/32-36
6. OC EDITION/A3/38-40

Figure 7-1. UDDF format for navigation aid data

```
                    1   0   0   1   1   1   1   1           QUOTIENT
1   1   0   0   1  |1   1   0   1   1   0   0   1       0   0   0   0
    X   O   R       1   1   0   0   1
                    ---------------------------------------------------
                    0   0   0   1   0   0   0   1
                                1   1   0   0   1
                                -----------------------------------------
                                0   1   0   0   0       0
                                    1   1   0   0       1
                                    -------------------------------------
                                    0   1   0   0       1   0
                                        1   1   0       0   1
                                        ---------------------------------
                                        0   1   0       1   1   0
                                            1   1       0   0   1
                                            -----------------------------
                                            0   1       1   1   1   0
                                                1       1   0   0   1
                                                -------------------------
                                                0       0   1   1   1
                              Remainder (CRC)
```

Figure 7-2. Data divided by GP to generate the CRC

```
                1   0   0   1   1   1   1   1
              ---------------------------------------------------
1 1 0 0 1    |  1   1   0   1   1   0   0   1       0   1   1   1
  X  O  R       1   1   0   0   1
              ---------------------------------------------------
                0   0   0   1   0   0   0   1
                            1   1   0   0   1
                          ---------------------------------------
                            0   1   0   0   0       0
                                1   1   0   0       1
                              -----------------------------------
                                0   1   0   0       1   1
                                    1   1   0       0   1
                                  -------------------------------
                                    0   1   0       1   0   1
                                        1   1       0   0   1
                                      ---------------------------
                                        0   1       1   0   0   1
                                            1       1   0   0   1
                                          -----------------------
                                            0       0   0   0   0
                       Remainder = 0 (no error)
```

Figure 7-3. Data with CRC divided by GP to establish whether corruption has occurred

Appendix A

THE GLOBAL POSITIONING SYSTEM (GPS)

1. BASIC CONCEPT

The Navigation System with Time And Ranging (NAVSTAR) Global Positioning System (GPS) is an all-weather, space-based navigation system, which has been designed primarily for the United States Department of Defense. In use since 1973, it became fully operational in 1994, allowing the worldwide and instantaneous determination of a vehicle's position and velocity (i.e. navigation) as well as the precise coordination of time.

2. SYSTEM ORGANIZATION

The Global Positioning System is made up of three major segments:

a) The *Control Segment* with ground-based equipment for monitoring the satellites and updating the information they transmit. As its name suggests, the Operational Control System (OCS) maintains and supports the rest of the system. It has three main activities — tracking, prediction, and uploading — and consists of a single Master Control Station (MCS), five monitor stations, and three ground antennas.

b) The *Space Segment* providing global coverage with four to eight simultaneously observable satellites above 15° elevation. This is accomplished by having satellites in six nearly circular orbits with an altitude of about 20 200 km above the earth over a period of approximately 12 hours. The number of operational satellites is 21 (plus three additional active spares), with an inclination of 55° and with four satellites per plane. The spare satellites are used to replace a malfunctioning "active" satellite.

c) The *User Segment*, comprising an unlimited number of receivers, which receive the satellite signals and calculate instantaneous position and other navigation information.

3. GPS SATELLITE SIGNAL STRUCTURE

3.1 The actual carrier broadcast by the satellite is a spread spectrum signal that makes it less subject to intentional (or unintentional) jamming. The spread spectrum technique is commonly used today by such diverse equipment as hydrographic positioning ranging systems and wireless Local Area Network (LAN) systems.

3.2 The key to the system's accuracy is the fact that all signal components are precisely controlled by atomic clocks. The Block II satellites have four on-board time standards: two rubidium and two caesium clocks. These highly accurate frequency standards, being the heart of GPS satellites, produce the fundamental *L*-band frequency of 10.23 MHz. Coherently derived from this fundamental frequency are two signals, the $L1$ and the $L2$ carrier waves, generated by multiplying the fundamental frequency by 154 and 120, respectively, thus yielding:

$L1$ = 1 575.42 MHz (19 cm), and

$L2$ = 1 227.60 MHz (24 cm).

3.3 These dual frequencies are essential for the elimination of the major source of error, i.e. ionospheric refraction. The pseudo-ranges that are derived from the measured travel time of the signal from each satellite to the receiver use two pseudo-random noise (PRN) codes that are modulated (superimposed) onto the two-base carrier waves. The first code is the C/A code (Coarse/Acquisition code), also designated as the Standard Positioning Service (SPS), which is available for civilian use. The C/A code, with an effective wavelength of 293.1 m, is modulated only on $L1$ and is purposely omitted from $L2$.

3.4 The second code is the P code (Precision code), also designated as the Precise Positioning Service (PPS), which has been reserved for use by the U.S. military and other authorized users. The P code, with an effective wavelength of 29.31 m, is modulated on both carriers $L1$ and $L2$.

4. GPS SATELLITE MESSAGE

4.1 In addition to the PRN codes, a data message is modulated onto the carriers consisting of:

a) satellite ephemerides,

b) ionospheric modelling coefficients,

c) status information,

d) system time and satellite clock bias, and

e) drift information.

4.2 The total message consisting of 1 500 bits is transmitted in 30 seconds with a data rate of 50 bits/sec. This message is subdivided into 5 subframes. One subframe is transmitted in 6 seconds and contains 10 words with 30 bits.

5. PSEUDO-RANGE AND CARRIER PHASE MEASUREMENTS

5.1 Figure A-1 shows pseudo-range and carrier phase measurements. Comparing the transmitted code from the satellite with a replica of it generated by the receiver results in the measurement of a time shift Δt, also called pseudo-range. Multiplying it by the velocity of light c (plus various corrections) results in a user-to-satellite distance.

5.2 The carrier of the signal emitted by the satellite is received (Doppler-shifted) by the receiver and compared with a generated carrier. The phase difference between both is the so-called carrier phase measurement.

5.3 This measurement is a subdivision of a wavelength of the signal, and the integer number of additional cycles making up the remainder of the distance is unknown. The integer cycle count is not observed but counted by the receiver. Every loss of lock leads to a loss of the number of cycles and produces a so-called cycle slip. Thus, since the initial value of n and the one after a cycle slip are unknown, phase measurements are ambiguous. This ambiguity (= integer number of cycles) has to be determined in the processing.

6. SYSTEM ASSURANCE TECHNIQUES

6.1 There are basically two methods for denying civilian users full use of the GPS system.

Selective Availability (S/A)

6.2 Primarily, this kind of denial has been accomplished by "dithering" the satellite clock frequency in a way that prevents civilian users from accurately measuring instantaneous pseudo-ranges (Dither-Process). This form of accuracy denial mainly affects single-receiver operations. When pseudo-ranges are differenced between two receivers, the dithering effect is largely eliminated, so that this navigation mode, proposed for example by the U.S. Coast Guard, will remain unaffected. The S/A has only been implemented in Block II satellites and has been in force intermittently since April 1990 at various levels of accuracy denial.

6.3 The second method is to truncate the transmitted navigation message so that the coordinates of the satellites cannot be accurately computed (Epsilon-Process). The error in satellite positions roughly translates to similar size position errors in the receiver.

Anti-spoofing

6.4 The design of the GPS system includes the ability to essentially "turn off" the P code or invoke an encrypted code (Y code) as a means of denying access to the P code to all but authorized users. The rationale for doing this is to keep adversaries from sending out false signals with the GPS signature, thereby creating confusion and causing users to misposition themselves. Access to the P code is only possible by installing on each receiver channel an Auxiliary Output Chip (AOC) which is available only on an authorized basis. Anti-spoofing affects many of the high-accuracy survey uses of the system.

7. GPS ABSOLUTE POSITIONING

7.1 Figure A-2 shows that if a three-dimensional position is to be determined, four pseudo-range measurements from different satellites have to be taken.

7.2 The extra measurement is to determine the clock offset between the very precise caesium clock of the satellite and the non-precise quartz clock of the receiver.

7.3 The following system of equations with four unknowns has to be solved:

$$(x1 - x)^2 + (y1 - y)^2 + (z1 - z)^2 + dT = (PR1)^2 \quad \text{(A-1)}$$
$$(x2 - x)^2 + (y2 - y)^2 + (z2 - z)^2 + dT = (PR2)^2 \quad \text{(A-2)}$$
$$(x3 - x)^2 + (y3 - y)^2 + (z3 - z)^2 + dT = (PR3)^2 \quad \text{(A-3)}$$
$$(x4 - x)^2 + (y4 - y)^2 + (z4 - z)^2 + dT = (PR4)^2 \quad \text{(A-4)}$$

GPS single-point absolute positioning accuracy

7.4 The civilian C/A code delivers a horizontal accuracy of 100 m (2 dRMS) if S/A is on, and 40 m (2 dRMS) if S/A is off. Vertical coordinates may be worse by a factor of two to three because of satellite-user geometry. Single-point positioning can be done in static as well as in kinematic (roving) mode.

7.5 It is interesting to note that the accuracies of the Russian satellite system GLONASS are in the same range as GPS but to our knowledge there is no S/A-type implementation on GLONASS.

8. POSITION ERRORS OF GPS

The following error sources of single-point positioning are possible:

a) satellite orbit;

b) satellite clock;

c) satellite code — selective availability (S/A);

d) receiver — resolution of the observation;

e) receiver — observation noise;

f) antenna — multipath effect; and

g) atmospheric refraction (ionosphere, troposphere).

9. DIFFERENTIAL GPS SURVEYING

9.1 The elimination of the various errors of single-point positioning can be achieved by forming "differences" between observations. The positioning of a static or roving user, relative to a fixed reference station with known WGS-84 coordinates, is called differential GPS positioning. Such a reference station can be used for an infinitely large number of users around it, say, in a radius of up to 100 km. The three-dimensional relative baseline vector, between the reference station and the user station, results from processing using GPS analysis software. Processing can be done in baseline or network mode.

9.2 Table A-1 shows the receiver noise and the theoretically expected differential position error when assuming a favourable satellite geometry (PDOP = 3). PDOP is a measure of satellite geometry: the smaller the number, the better.

10. CHOICE OF GPS SURVEYING TECHNIQUE

10.1 Depending on the desired coordinate accuracies, one can choose between the following processing techniques:

a) Metre and submetre accuracies —

Differential phase-smoothed pseudo-range processing. Here the receiver costs are moderate (< US $10 000). Real-time surveying seems to be more robust than using pure phase measurements.

b) Centimetre accuracies —

Carrier phase-based approach. The necessary equipment and analysis software have a significantly higher price, mainly due to the use of dual frequency receivers. Different observation strategies are possible: static, rapid or fast static, pseudo-kinematic, stop-and-go, semi-kinematic and kinematic mode. These are explained in Table A-2.

GPS surveying modes and accuracy

10.2 Table A-2 shows the various GPS surveying techniques and the corresponding resulting baseline accuracies.

Table A-1. Observation noise and error propagation

C/A code observations	*Receiver noise*	*Differential position error (PDOP = 3)*
Pseudo-range	0.2 to 5 m	0.6 to 15 m
Carrier phase	0.2 to 2 mm	0.6 to 6 mm

Table A-2. GPS surveying modes and accuracy

Mode	*Characteristics*	*Accuracy*
Static	Long observation time (hours to days) Long baselines (say > 100 km)	±0.1 mm to ±1 ppm
Rapid static Fast static	Short observation time (5 to 30 minutes) Short baselines (say < 10 km) Dual frequency receivers preferable	±(5 mm + 1 ppm)
Pseudo-kinematic	Short observation time (few minutes) Reoccupations of stations necessary	±(5 mm + 1 ppm)
Stop-and-go Semi-kinematic	Short observation time (few minutes) Maintain lock between stations	±(5 mm ± 1 ppm)
Kinematic	No stopping required Sophisticated software needed	±(1 to 5 cm ± 1 ppm)

Differential GPS real-time positioning

10.3 Since mid-1994, differential GPS (DGPS) real-time surveying has been offered by several companies. The real-time aspect came about not because of any necessity for immediate results, but in order to carry out quality control in the field.

10.4 Figure A-3 demonstrates the principle of differential GPS real-time positioning, i.e.:

a) positioning of a (roving) user relative to a reference station with known coordinates;

b) determination of GPS pseudo-range and/or carrier phase corrections at the reference station;

c) transmission of the corrections to the mobile user by telemetry; and

d) quality and error control by monitor stations.

11. DIFFERENTIAL CORRECTIONS

11.1 Three types of corrections sent out by the reference station are possible:

a) position or pseudo-range corrections,

b) carrier-smoothed pseudo-range corrections, and

c) carrier phase corrections.

Differential pseudo-range corrections

11.2 A differential pseudo-range correction is the difference between the observed and calculated (from known reference station coordinates and transmitted satellite ephemerides) pseudo-range at the reference station.

11.3 The advantage over formerly used position corrections is that biases due to different satellite tracking scenarios at reference and user stations are avoided. The possible real-time surveying accuracy is 3 to 6 m.

Differential carrier-smoothed pseudo-range corrections

11.4 The principle is the same as for the differential pseudo-range corrections, but now the carrier phases are used to smooth the pseudo-ranges in a filter. This results in higher positioning accuracies, and antenna multipath is eliminated to a large extent. No repair of cycle slips is necessary, only the necessity to detect them. The possible real-time surveying accuracy is 0.6 to 2 m.

Differential carrier phase corrections

11.5 This type of correction involves the calculation and transmission of the corrections at a known reference station. This is a new development with the following characteristics.

a) Advantages:

— navigation in the centimetre range;

— smaller amount of telemetry data compared to raw data;

— reduction of time sensitivity; and

— reduced computational burden for roving user.

b) Disadvantages:

— necessity: Carrier phase ambiguity resolution on-the-fly;

— dependence of the user with respect to the integrity and reliability of the calculation at the reference station; and

— error detection more difficult.

11.6 The possible real-time surveying accuracy is 1 to 5 cm.

12. ACCURACY OF GPS

Figure A-4 shows the GPS (absolute) and DGPS (differential) navigation and surveying accuracies achievable, along with corresponding statistical distributions.

13. GPS DIFFERENTIAL POSITIONING TECHNIQUES

13.1 GPS differential positioning techniques have the following characteristics.

a) Advantages:

— all-weather 24-hour positioning capability;

— various levels of accuracy are possible, depending on the available hardware and software; and

— easy to use.

b) Disadvantages:

— no line-of-sight between target (user station) and the reference station required; however, line-of-sight to at least four satellites is needed; and

— since certain signal obstruction by buildings, trees, etc. may occur, a certain amount of conventional surveying must still be carried out.

13.2 Only ellipsoid height differences can be determined by DGPS. In order to get orthometric heights, one has to use a geoid model of appropriate accuracy.

FIGURES FOR APPENDIX A

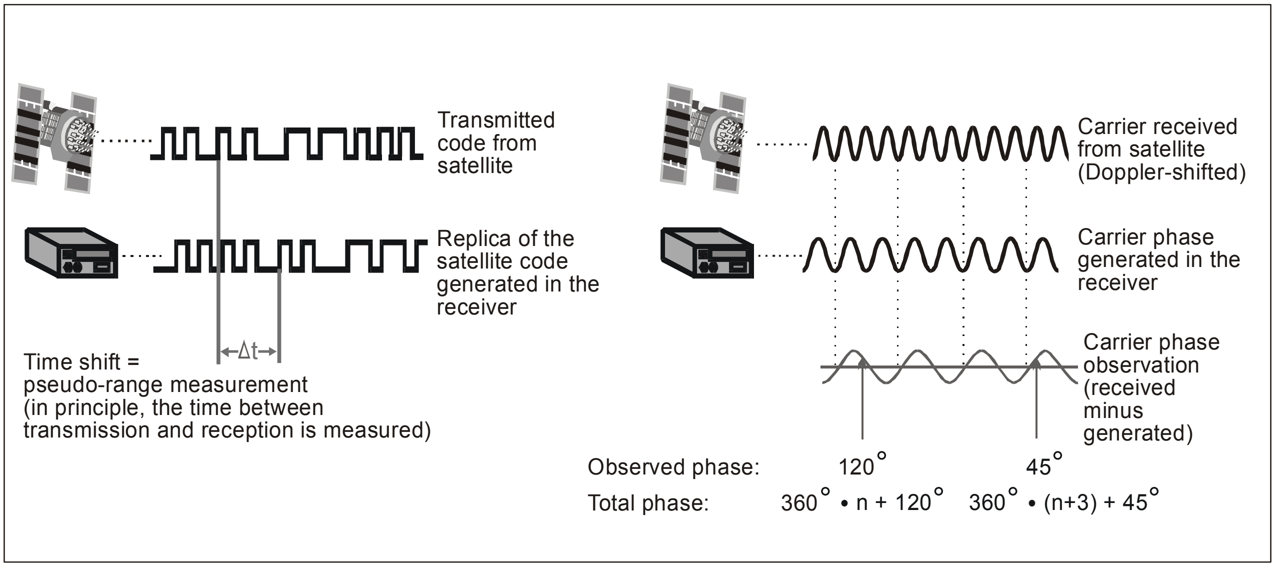


Figure A-1. Pseudo-range and carrier phase measurement

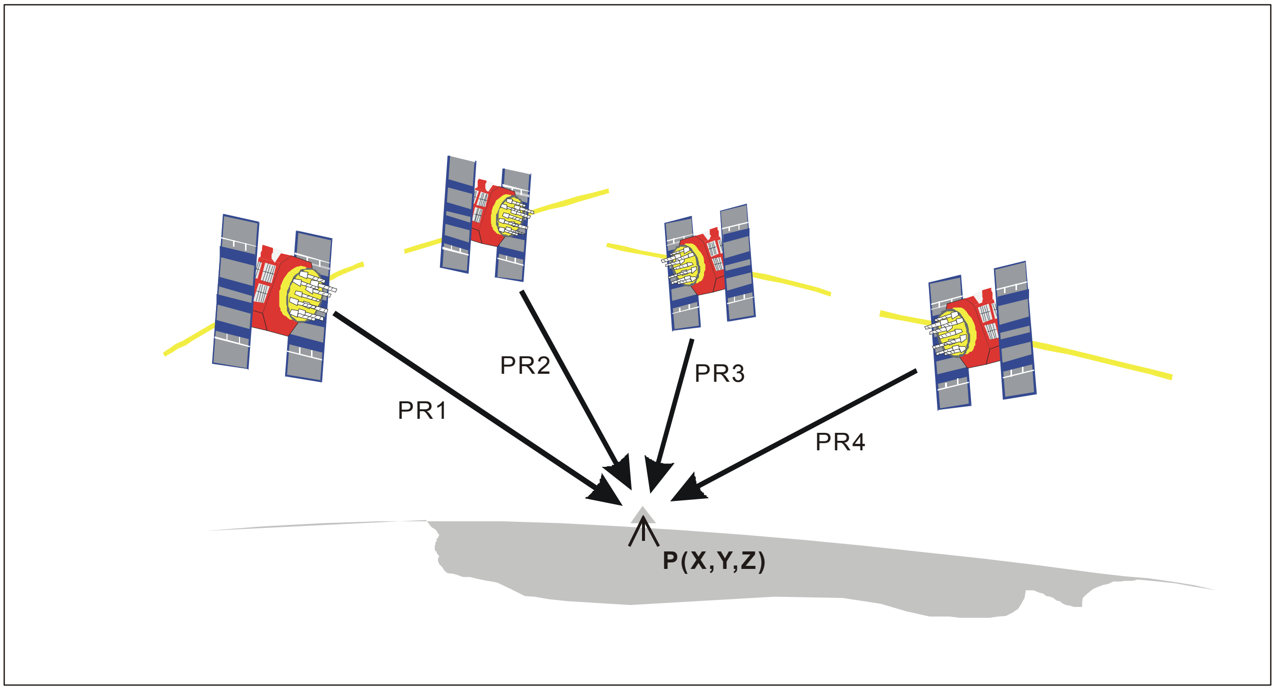


Figure A-2. Principle of GPS absolute positioning

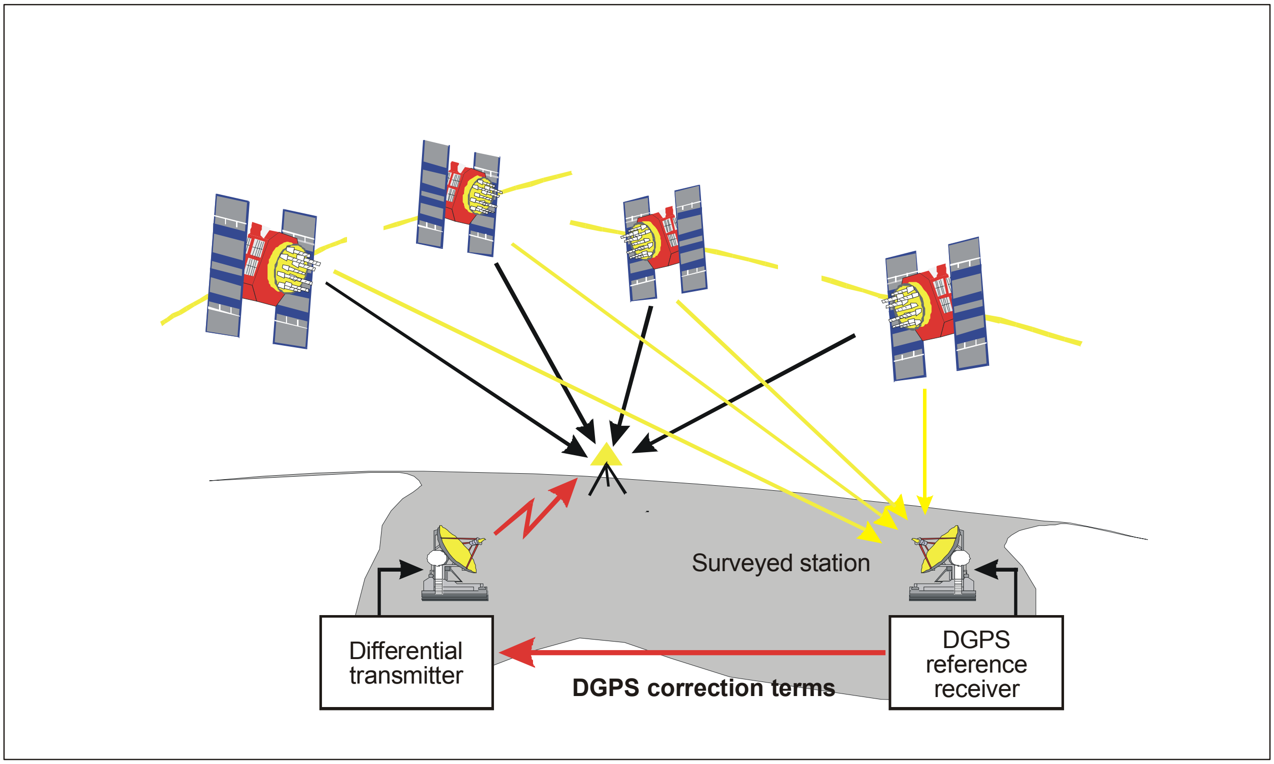


Figure A-3. Differential GPS real-time positioning

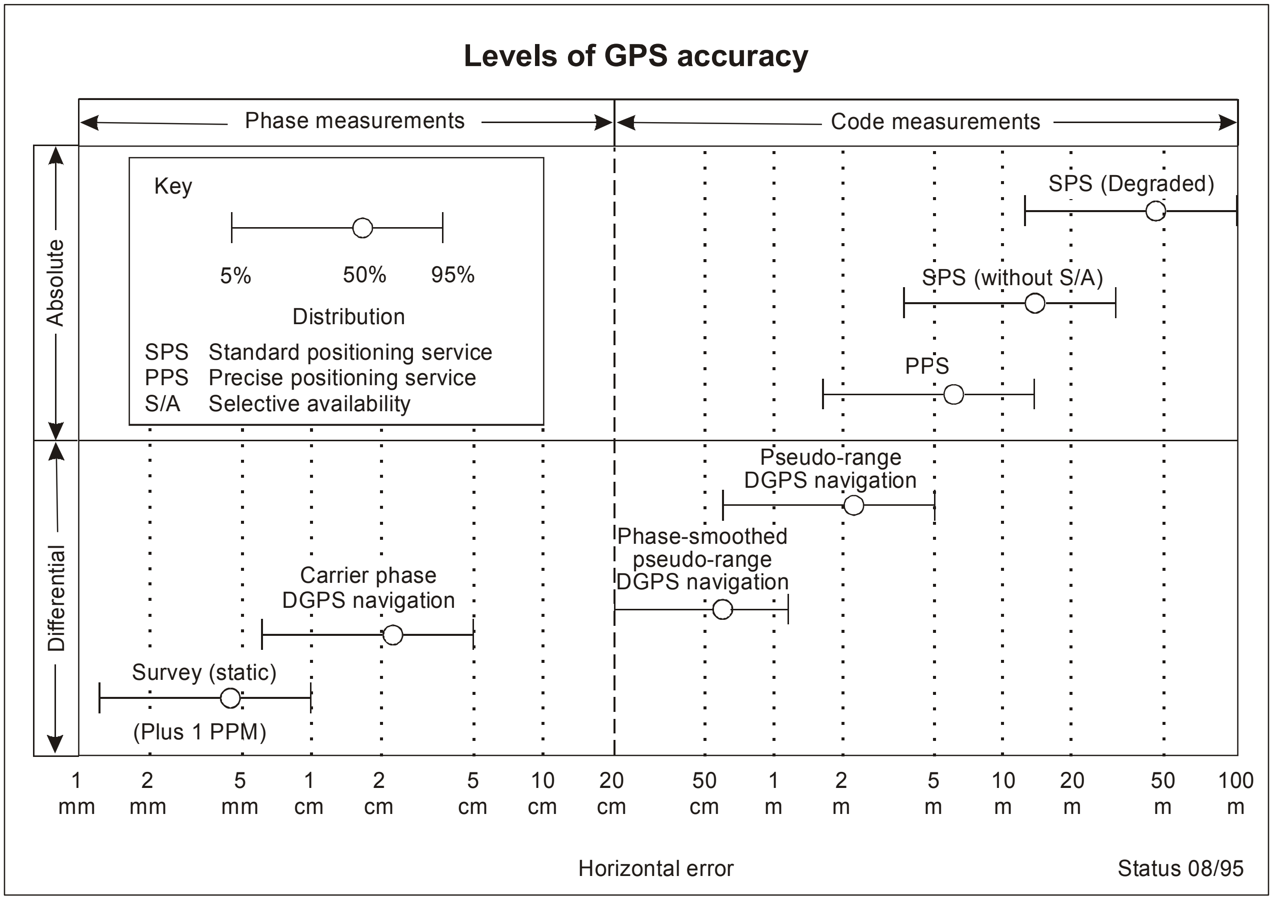


Figure A-4. Accuracy of GPS

Appendix B

PRINCIPLES OF GEODESY

1. DEFINITION OF GEODESY

"Geodesy is the science concerned with the study of the shape and size of the earth in the geometric sense as well as with the form of the equipotential surfaces of the gravity potential."

1.1 This definition was penned in 1880 by Friedrich R. Helmert, one of the main proponents of geodesy of the nineteenth century.

1.2 The following represents a more up-to-date description given by the Committee on Geodesy of the U.S. National Academy of Sciences (1978).

a) Establishment and maintenance of national and global three-dimensional geodetic networks.

b) Measurement and analyses of geodynamic phenomena (earth rotation, earth tides, crustal movements, etc.).

c) Determination of the earth's gravity field.

These three items also include changes with time.

1.3 The time-related monitoring of coordinates and of the gravity field is the new aspect.

Why does geodesy deal with the gravity field?

1.4 Every geodetic measurement is a function of the gravity field. For example, by putting an instrument into the horizontal plane by using spirit bubbles, it aligns its vertical axis with the local plumb-line (local gravity vector) which, unfortunately, may vary from point to point.

1.5 In defining heights, an equipotential surface of the gravity field must be used as vertical reference. (Where does water flow?)

2. FIGURE OF THE EARTH AND REFERENCE SURFACES

2.1 The figure of the earth was approximated first by a sphere and later by an ellipsoid. Whereas these approximations are of geometrical character, the geoid represents a dynamical reference surface — a certain equipotential surface of the earth's gravity field.

The earth as a sphere

2.2 Various opinions on the form of the earth prevailed in the past, e.g. the notion of an earth disk encircled by Oceanus. Homer's Illiad (approximately 800 B.C.), Pythagoras (approximately 580–500 B.C.) and his school, and Aristotle (384–322 B.C.), among others, expressed belief in the spherical shape.

2.3 The founder of scientific geodesy is Eratosthenes (276–195 B.C.) of Alexandria who, assuming the earth was spherical, deduced from measurements a radius for the earth with 2% uncertainty (left-hand side of Figure B-1).

2.4 The principle of arc measurements developed by him was still applied in modern ages: from geodetic measurements, the length ΔG of a meridian arc is determined. Astronomical observations furnish the associated central angle γ. The radius of the earth is then given by $R = \Delta G/\gamma$.

2.5 The arc measurements in middle ages are characterized by fundamental advances in instrumentation technology. Arc measurements and early triangulations are shown on the world map of Figure B-1.

The earth as an ellipsoid

2.6 Towards the end of the seventeenth century, Newton demonstrated that the concept of a truly spherical earth was inadequate as an explanation of the equilibrium of the ocean surface. He argued that because the earth is a rotating planet, the forces created by its own rotation would tend to force any liquids on the surface to the equator. He showed, by means of a simple theoretical model, that hydrostatic equilibrium would be maintained if the equatorial axis of the earth were longer than the polar axis. This is equivalent to the statement that the body is flattened towards the poles.

2.7 Flattening is defined by $f = (a - b)/a$ where a is the semi-major and b is the semi-minor axis of the ellipsoid.

2.8 In the eighteenth and nineteenth centuries, ellipsoids were defined which were best fitted to a certain region of the earth (Figure B-2). These local ellipsoids still provide the geometrical reference for the horizontal coordinates of various national geodetic (triangulation) networks.

2.9 Table B-1 shows examples of the ellipsoidal parameters of various ellipsoids. Note that the East Europeans (the former U.S.S.R) based their horizontal coordinates on a triaxial ellipsoid (Krassovsky).

The earth as a geoid

Definition of ***Geoid.*** The equipotential surface in the gravity field of the Earth which coincides with the undisturbed mean sea level (MSL) extended continuously through the continents.

Note.— The geoid is irregular in shape because of local gravitational disturbances (wind tides, salinity, current, etc.) and the direction of gravity is perpendicular to the geoid at every point.

Definition of ***Geoid undulation.*** The distance of the geoid above (positive) or below (negative) the mathematical reference ellipsoid.

Note.— In respect to the World Geodetic System — 1984 (WGS-84) defined ellipsoid, the difference between the WGS-84 ellipsoidal height and orthometric height represents WGS-84 geoid undulation.

2.10 Laplace (1802), Gauss (1828), Bessel (1837) and others had already recognized that the assumption of an ellipsoidal earth model was not tenable when compared to high-accuracy observations. One could no longer ignore the deflection of the physical plumb-line, to which measurements refer, from the ellipsoidal normal (deviation of the vertical, see Figure B-3). By an adjustment of several arc measurements for the determination of the ellipsoidal parameters a and f, contradictions arose which exceeded by far the observational accuracy.

2.11 Although Listing (1873) had assigned the name for the geoid, Helmert (1880, 1884) made the transition to the current concept of the figure of the earth. Here the deflections of the vertical are also taken into account in the computation of the ellipsoidal parameters.

2.12 The determination of the geoid has been, for the last hundred years, a major goal of geodesy. Its importance increased recently with the new concept of replacing the measurements of spirit levelling by GPS space observations and the use of precise geoid heights. Other global considerations require a unified vertical reference, i.e. a geoid determination with centimetre or even millimetre accuracy. This remains a challenge for geodesy in the coming years.

2.13 There are difficulties in defining a geoid such as sea-surface topography, sea-level rise (melting of the polar ice caps) and density changes (earthquakes).

3. COORDINATE SYSTEMS AND REFERENCE ELLIPSOIDS

3.1 The geodetic glossary gives the following general geodetic definitions.

a) *Coordinate.* One of a set of N numbers designating the location of a point in N-dimensional space.

b) *Coordinate system.* A set of rules for specifying how coordinates are to be assigned to points (origin) set of axes.

3.2 There are three types of coordinate systems: local, geocentric (earth-fixed) and Cartesian, or ellipsoidal.

Local coordinate system

3.3 Due to historical developments, national departments of survey computed, ellipsoids best fitted to their State to provide the basis for mapping. The origin and orientation of a coordinate system are arbitrary, but often “the ball under the cross on top of the tower of a specific church” served as the zero-point (or origin) of a national coordinate system (e.g. Soldner’s coordinate system in Bavaria with the Munich cathedral “Liebfrauendom” as origin). The national ellipsoids are the geometric reference surfaces only for horizontal coordinates.

Geocentric (earth-fixed) Cartesian system (X, Y, Z)

3.4 As a fundamental terrestrial coordinate system, an earth-fixed spatial Cartesian system (X, Y, Z) is introduced whose origin is the earth’s centre of mass “S” (geocentre —

centre of mass including the mass of the atmosphere (see Figure B-4)). The Z-axis coincides with the mean rotational axis of the earth (polar motion).

3.5 The mean equatorial plane perpendicular to this axis forms the X-Y plane. The X-Z plane is generated by the mean meridian plane of Greenwich. The latter is defined by the mean rotational axis and the zero meridian of the Bureau International de l'Heure (BIH)-adopted longitudes ("mean" observatory of Greenwich). The Y-axis is directed so as to obtain a right-handed system. The introduction of a mean rotational axis is necessary because in the course of time, the rotation changes with respect to the earth's body. This applies to the position of the earth's rotation axis (polar motion) and to the angular velocity of the rotation.

Ellipsoidal geographic coordinates

3.6 As Figure B-5 shows, the earth's surface may be closely approximated by a rotational ellipsoid with flattened poles (height deviation from the geoid < 100 m). As a result, geometrically defined ellipsoidal systems are frequently used instead of the spatial Cartesian coordinate system.

3.7 The rotational ellipsoid is created by rotating the meridian ellipse about its minor axis. The shape of the ellipsoid is therefore described by two geometric parameters, the semi-major axis a and the semi-minor axis b. Generally, b is replaced by a smaller parameter which is more suitable: the (geometrical) flattening f.

$$f = (a - b)/a$$

3.8 Further definitions:

a) *Origin.* Earth's centre of mass.

b) *Geographic (geodetic) latitude* ϕ. Angle measured in the meridian plane between the equatorial (X, Y) plane and the surface normal at P.

c) *Geographic (geodetic) longitude* λ. Angle measured in the equatorial plane between the zero meridian (X-axis) and the meridian plane of P.

Spatial ellipsoidal coordinate system

3.9 For the spatial determination of points on the physical surface of the earth (or in space) with respect to the rotational ellipsoid, the height h above the ellipsoid is introduced in addition to the geographic coordinates ϕ, λ. The ellipsoidal height h is measured along the surface normal (Figure B-6).

3.10 The spatial ellipsoidal coordinates ϕ, λ, h are designated as geodetic coordinates. The point Q on the ellipsoid is obtained by projecting the surface (or space) point P along the ellipsoidal normal. A point in space is defined by ϕ, λ, h and the shape of the ellipsoid (a, f).

3.11 A standard earth model as a geodetic reference body should guarantee a good fit to the earth's surface and to the external gravity field, but it should also possess a simple principle of formation.

3.12 In this respect, the rotational ellipsoid, already introduced as a geometric reference surface, is well suited. In addition to the semi-major axis a and the flattening f as geometric parameters, the total mass M and the rotational angular velocity ω as physical parameters are introduced. The gravity field is then formed as a result of gravitation and rotation.

3.13 If we now require the surface of this ellipsoid to be a level surface of its own gravity field, then, according to Stokes Theorem, the gravity field is uniquely defined in the space exterior to this surface. This body is known as a level (or equipotential) ellipsoid. Additionally, the geocentric gravitational constant GM and the dynamic flattening $\overline{C}_{2,0}$ (second order zonal harmonic of an earth gravity model) are given. If the ellipsoidal parameters are given those values which correspond to the real earth, then this yields the optimum approximation to the geometry of the geoid and to the external gravity field: mean earth ellipsoid.

3.14 Table B-1 lists the reference ellipsoids and their constants a, f associated with local geodetic datums which are tied to WGS-84 through datum transformation constants and/or multiple regression equations.

4. GEODETIC DATUM

Definitions

4.1 The terminology required to describe the geodetic datum problem is rather complex and has developed over more than a century. In order to avoid confusion and misunderstanding, care must be taken to use the various terms precisely.

4.2 The following definitions have been adopted by the international geodetic community:

a) *Geodetic reference system.* Conceptual idea of an earth-fixed Cartesian system (X, Y, Z).

b) *Geodetic reference frame.* Practical realization of a geodetic reference system by observations.

Table B-1. Reference ellipsoid names and constants (WGS-84 minus local geodetic datum)

Reference ellipsoid name	*ID code*	*a (m),* *Δa (m)*	f^{-1}, $\Delta f \times 10^{-4}$
Airy 1830	AA	6377563.396, 573.604	299.3249646, 0.11960023
Australian National	AN	6378160, –23	298.25, –0.0001204237
Bessel 1841			
Ethiopia, Indonesia, Japan, Korea	BR	6377397.155, 739.845	299.1528128, 0.1003748283
Namibia	BN	6377483.865, 653.135	299.1528128, 0.1003748283
Clarke 1866	CC	6378206.4, –69.4	294.9786982, –0.3726463909
Clarke 1880*	CD	6378249.145, –112.145	293.465, –0.547507137
Everest			
Brunei and E. Malaysia (Sabah and Sarawak)	EB	6377298.556, 838.444	300.8017, 0.2836136834
India 1830	EA	6377276.345, 860.655	300.8017, 0.2836136834
India 1956**	EC	6377301.243, 835.757	300.8017, 0.2836136834
W. Malaysia and Singapore 1948	EE	6377304.063, 832.937	300.8017, 0.2836136834
W. Malaysia 1969**	ED	6377295.664, 841.336	300.8017, 0.2836136834
Geodetic Reference System 1980	RF	6378137, 0	298.257222101, -1.6193×10^{-7}
Helmert 1906	HE	6378200, –63	298.3, 0.004807957
Hough 1960	HO	6378270, –133	297, –0.14192702
International 1924 (Hayford)	IN	6378388, –251	297, –0.14192702
Krassovsky 1940	KA	6378245, –108	298.3, 0.004807957
Modified Fischer 1960	FA	6378155, –18	298.3, 0.004807957
South American 1969	SA	6378160, –23	298.25, –0.000812042
WGS-72	WD	6378135, 2	298.26, 0.000312108
WGS-84	WE	6378137, 0	298.257223563, 0

* As accepted by NIMA.
** Through adoption of a new yard to metre conversion factor in the reference State.

4.3 It is important to make a distinction between a reference system and a reference frame. A reference system is the conceptual idea of a particular coordinate system (theoretical definition). A reference frame is the practical realization of a reference system by observations and measurements (which have errors). Practical surveying is only concerned with reference frames, but the underlying concepts of a specific reference frame are of fundamental importance.

a) Global GRS:

— origin: earth's centre of mass;

— Z-axis: coincides with mean rotational axis of earth;

— X-axis: mean meridian plane of Greenwich and perpendicular to the Z-axis; and

— Y-axis: orthogonal.

b) Local GRS:

— origin and orientation of axes are "arbitrary".

c) Geodetic datum:

— minimum set of parameters required to define location and orientation of the local system with respect to the global reference system/frame.

4.4 Furthermore, it is important to distinguish between global and local reference frames. Looking at the entire set of possible reference frames located in the body of the earth, there is only one truly global reference system. The origin of a global reference system coincides with the centre of the earth, the Z-axis should coincide with the mean rotational axis of the earth and the X-axis is contained in the mean meridian plane of Greenwich, perpendicular to the Z-axis. The Y-axis is orthogonal to both the X- and Z-axes (right-hand system).

4.5 A geodetic datum is expressed in terms of the set of transformation parameters which are required to define the location and orientation of the local frame with respect to the global frame.

Note.— The term "datum" is often used when one actually means "reference frame".

4.6 A distinction must be made between a Cartesian datum and an ellipsoidal datum.

4.7 A Cartesian datum is defined by a set of three shifts (ΔX, ΔY, ΔZ); three rotations (α, β, γ); and a scale factor (μ). These seven parameters are needed to relate two Cartesian three-dimensional reference frames.

4.8 Because the earth is a curved surface approximated by an ellipsoid, navigators usually work in geographical coordinates (latitude, longitude). In order to define geographical coordinates, the shape of the so-called reference ellipsoid also has to be considered. The shape of an ellipsoid is defined by its semi-major and semi-minor axes, i.e. two additional parameters are required. These two additional parameters constitute the difference between a Cartesian and an ellipsoidal datum. Thus, an ellipsoidal datum is defined by nine transformation parameters.

4.9 Rule of thumb: ellipsoidal datum = Cartesian datum + shape of earth ellipsoid.

5. TRANSFORMATIONS

5.1 A geodetic datum transformation is a mathematical rule used to transform surveyed coordinates given in a Reference Frame 1 into coordinates given in Reference Frame 2. The mathematical rule is a function of the set of necessary datum transformation parameters.

5.2 The nine parameters:

— translation (shift) of the origin: ΔX, ΔY, ΔZ;

— rotation angles: εx, εy, εz;

— scale factor μ; and

— change in ellipsoidal semi-major axis Δa and flattening Δf,

define the location and orientation of a (local) coordinate system with respect to a global reference frame. These parameters are needed for a computational coordinate transformation using Helmert's formula.

6. HEIGHT

What is "height"?

6.1 Usually, the implicit imagination behind the term "height" is the answer to the question: Where does water flow? Physically, we consider a lake where water is in rest as a surface of equal heights. More specifically, it is the equipotential surface in the gravity field of the earth. Moving on such a surface means no work is carried out; no forces are acting on it. Thus, the definition of a height with such a physical meaning cannot be defined geometrically nor can the reference surface (zero surface) be the ellipsoid's geometrical surface.

Geodetic networks

6.2 The application of differential GPS satellite observations delivers:

a) horizontal WGS-84 coordinates: ellipsoidal latitude ϕ and longitude λ; and

b) vertical WGS-84 coordinates: ellipsoidal height h.

6.3 The ellipsoidal height does not have a physical meaning. It is a geometric quantity that does not indicate a level surface (i.e. it does not indicate the direction of the flow of water). Geodetic networks consist, in general, of geometrically defined and referred ellipsoidal latitude and longitude, whereas national heights refer to the geoid ("mean sea level") as zero surface.

The geoid as reference surface for heights

6.4 The geoid can be considered as an idealized ocean extending under the continents (Figure B-7). It is a particular equipotential gravity surface of the earth coinciding with approximately two-thirds of the earth's surface. There is only one geoid.

6.5 The geoid is realized in practice by observing "mean sea level" at tide gauges at the coasts over a certain time period. However, there are complications brought about by wind, salinity, currents, etc., that produce deviations from the geoid of up to 2 m ("sea surface topography"). This means that the zero point and, consequently, the heights of different national networks may differ by similar magnitudes.

6.6 Heights above the geoid are called orthometric heights H. The relationship between an ellipsoidal height h and H is given by:

$$H = h - N$$

where N is the geoid undulation.

Vertical datum

6.7 WGS-84 is a three-dimensional reference frame coordinated in X, Y, Z or in ϕ, λ, h. The parameter h is the (geometric) height above the WGS-84 ellipsoid.

6.8 In aviation, heights (flight level) are defined by atmospheric pressure. All aircraft are therefore equipped with baro-altimeters. One has to be very careful when dealing with heights. The differences between the different zero points of national vertical networks may vary up to 3 metres! Presently, there is a worldwide effort to come up with a unified height system. It is hoped that this zero surface (namely the geoid) can be determined worldwide to an accuracy < 20 cm by using satellite altimetry.

6.9 Table B-2 shows vertical datum differences.

Table B-2. Reference surface differences with respect to the geoid

State/reference surface		*Difference to geoid (cm)*
Australia:	Mainland	–68
	Tasmania	–98
England		–87
United States:	NAVD 29	–26
	NAVD 88	–72
	NAVD 88, East	–38
Germany		4

FIGURES FOR APPENDIX B

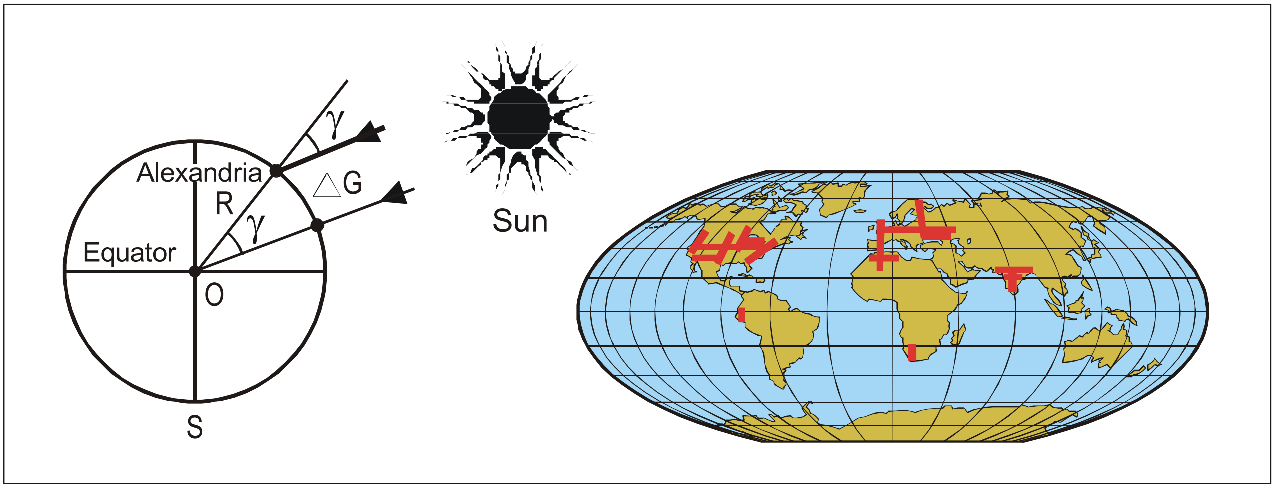


Figure B-1. The earth as a sphere, derived from arc measurements

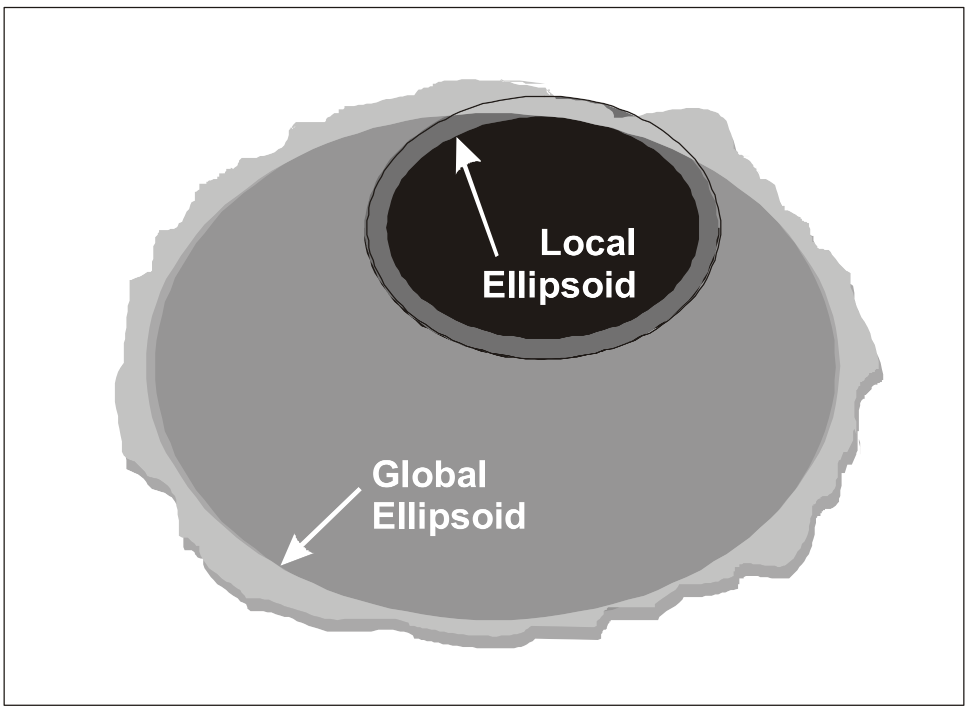


Figure B-2. Local ellipsoids are best fitted to the specific State

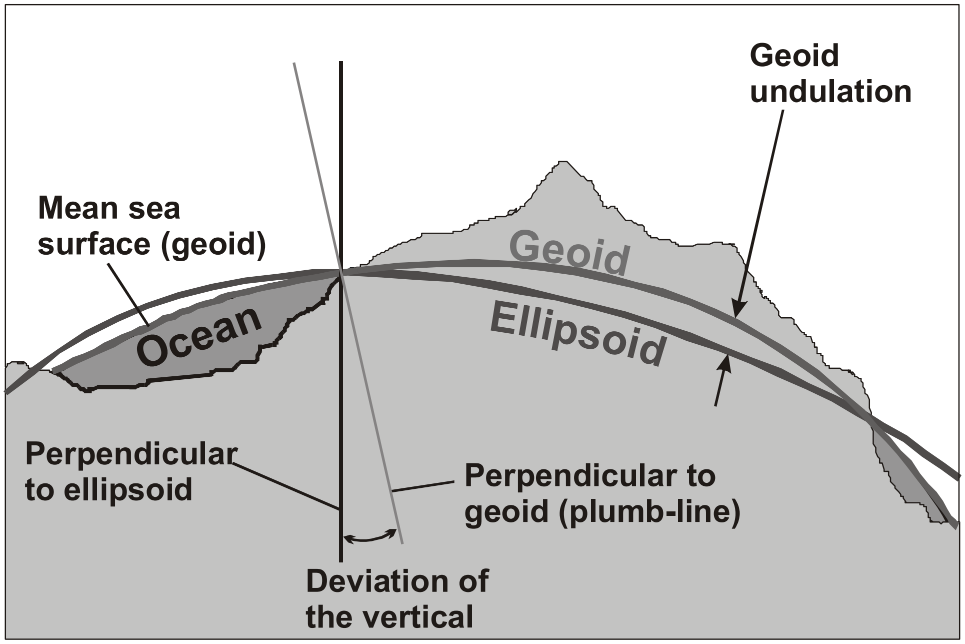


Figure B-3. The earth as a geoid

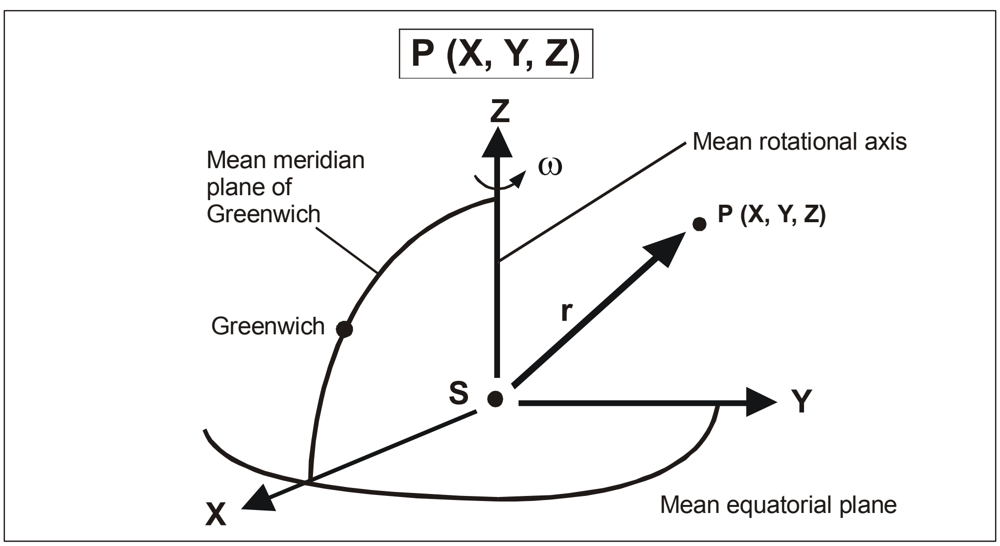


Figure B-4. Earth-fixed spatial Cartesian system (X, Y, Z)

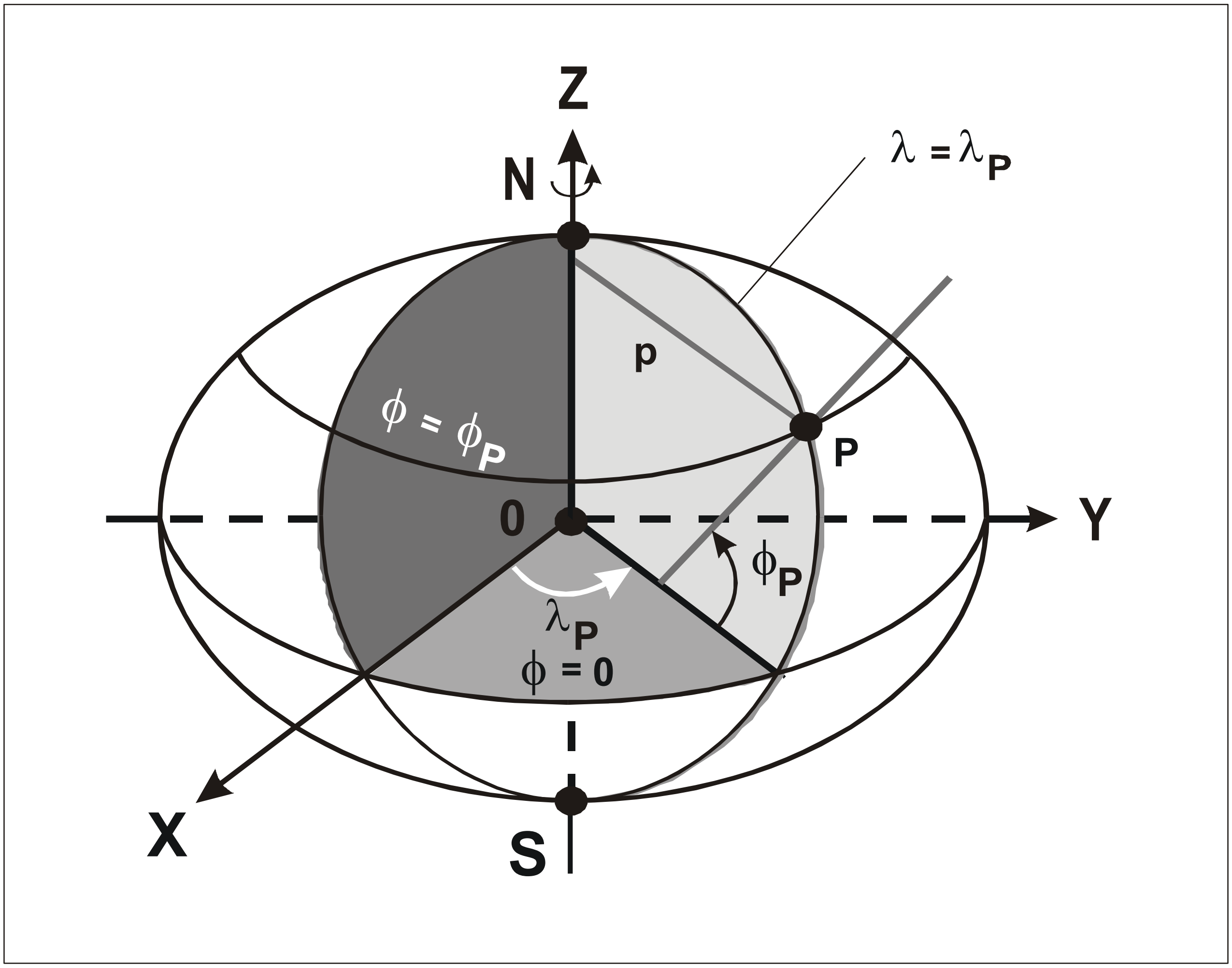


Figure B-5. Ellipsoidal geographic coordinates

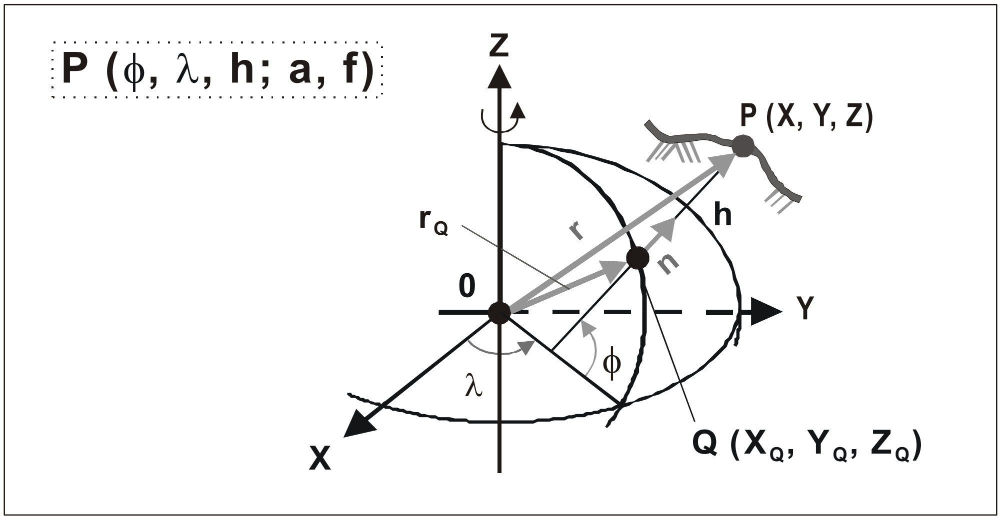


Figure B-6. Spatial ellipsoidal coordinate system

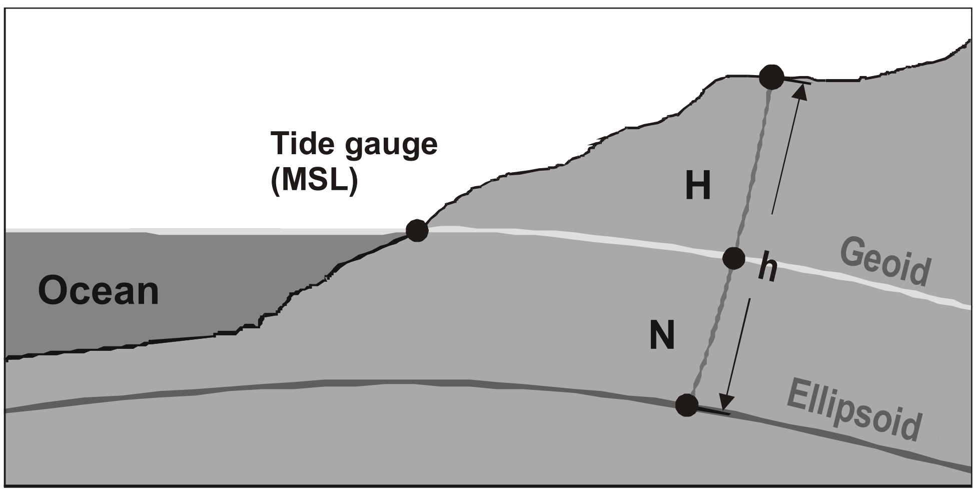


Figure B-7. The geoid as reference surface for heights

Appendix C

THE INTERNATIONAL TERRESTRIAL REFERENCE SYSTEM (ITRS)

1. The most precise geodetic measuring techniques for long base lines are, at present, satellite laser ranging (SLR) and very long base-line interferometry (VLBI). Both techniques guarantee a precision of 1 to 3 cm over distances up to about 5 000 km. Global networks of up to 70 SLR stations and up to 81 VLBI stations were established for continuous observation and data collection. Since 1987, a new International Earth Rotation Service (IERS) has been operating, making use of SLR and VLBI results predomi-nantly, and producing a new global set of X, Y, Z coordinates every year by combining various SLR and VLBI solutions.

2. The precise SLR technique has led to a precise worldwide terrestrial coordinate system, called the International Terrestrial Reference System (ITRS). The ITRS is maintained by the IERS and the realization of the ITRS is the International Terrestrial Reference Frame (ITRF).

3. Plate tectonic movement was incorporated in that coordinate system using results of recent measurements and a global geophysical model. Thus, it is a model with changing coordinates due to movements of tectonic plates on which the ground stations are located. However, this reference system provides the fundamental position of the earth to within 10 cm and the orientation of the axes to correspondingly high accuracies. Since 1988, the IERS has defined the mean spin axis, the IERS Reference Pole (IRP), the zero meridian and the IERS Reference Meridian (IRM).

4. The maintenance of a datum at this level of accuracy requires constant monitoring of the rotation of the earth, the motion of the IRP and the movement of the plates of the crust of the earth on which the ground stations are located. The current definition of ITRF is known as ITRF 97, which means the computation of the ITRF coordinates at epoch 1997.0.

Appendix D

DATUM TRANSFORMATION FORMULAE

1. GENERAL

1.1 The general task of a datum transformation can be expressed as follows:

Given: a point with spatial ellipsoidal coordinates (geodetic latitude ϕ and geodetic longitude λ, ellipsoidal height h) referring to a local ellipsoid with semi-major axis a and flattening f):

Find: the geodetic latitude ϕ, longitude λ and ellipsoidal height h referring to the WGS-84 ellipsoid.

This is illustrated in Figure D-1.

1.2 The following three transformations are explained in more detail in this appendix:

a) Helmert's Formula,

b) Standard Molodensky Formula, and

c) Multiple Regression Equation.

1.3 The clear advantage of computational transformations over WGS-84 surveying is the minimal effort required in using appropriate software and known datum parameters.

Note.— All datum transformations require the use of the ellipsoidal height h *in the local system which is* h = H + N *with* H *being the orthometric height and* N *being the geoid undulation (height). In general, only the orthometric height is known (and found also in maps). The geoid undulation (height) has to be taken from a digital model (if available).*

1.4 An investigation was made checking the effect of an unknown (orthometric) height on the transformed latitude and longitude of a point using the Helmert transformation formula. By assigning heights ranging from 0 m to 8 000 m, it was concluded that the effect on both latitude and longitude was negligible (less than 15 cm at 8 000 m). Consequently, for a point of known latitude and longitude but unknown (orthometric) height, an arbitrary height of 0 m could be assigned without significantly affecting the transformation.

2. HELMERT'S FORMULA

2.1 The application of Helmert's formula requires a three-step approach if the input coordinates are given in spatial ellipsoidal coordinates ϕ, λ, h. If the input coordinates are already given in rectangular coordinates X, Y, Z of a local system, then proceed to Step 2.

2.2 *Step 1.* Transformation from the spatial ellipsoidal coordinates ϕ, λ, h of the local ellipsoid into rectangular coordinates X, Y, Z of the local system.

$$(\phi, \lambda, h)_{Local} \rightarrow (X, Y, Z)_{Local}$$

$$X = (v + h)\cos\phi\cos\lambda$$

$$Y = (v + h)\cos\phi\sin\lambda$$

$$Z = (v(1 - e^2) + h)\sin\phi$$

v = radius of curvature in the prime vertical

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

a = semi-major axis of ellipsoid

e = eccentricity of ellipsoid ($e^2 = f(2 - f)$)

f = flattening of ellipsoid

Note.— See Table B-1 for a list of reference ellipsoids and parameters.

2.3 *Step 2.* Depending on the availability and reliability of datum parameters, this transformation may use only the three shifts of origin, or the three shifts of origin and the scale parameter, or all seven parameters including the rotation angles.

$$\begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{WGS\text{-}84} = \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{Local} + \underbrace{\begin{bmatrix} \mu & +\varepsilon_Z & -\varepsilon_Y \\ -\varepsilon_Z & \mu & +\varepsilon_X \\ +\varepsilon_Y & -\varepsilon_X & \mu \end{bmatrix}}_{\text{Rotation angles and scale factor}} \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{Local} + \underbrace{\begin{bmatrix} \Delta X \\ \Delta Y \\ \Delta Z \end{bmatrix}}_{\text{Shift of origin}}$$

Note.— See Table D-1 for a list of WGS-84 transformation parameters.

2.4 *Step 3.* The back transformation from WGS-84 Cartesian coordinates into spatial ellipsoidal WGS-84 coordinates ϕ, λ, h is performed. It should be noted that the back transformation is carried out below in an iterative manner. However, the development converges quickly due to $h < n$.

$$(\phi, \lambda, h)_{WGS\text{-}84} \leftarrow (X, Y, Z)_{WGS\text{-}84}$$

$$\phi = \arctan \frac{Z}{\sqrt{X^2 + Y^2}} \left(1 - e^2 \frac{v}{v + h}\right)^{-1}$$

$$\lambda = \arctan \frac{Y}{X}$$

$$h = \frac{\sqrt{X^2 + Y^2}}{\cos\phi} - v$$

2.5 The exact back transformation formula presented below is provided in the "Exact Conversion of Earth-Centred Earth-Fixed Coordinates to Geodetic Coordinates" by J. Zhu, *Journal of Guidance, Control and Dynamics,* Vol. 16, No. 2, March-April 1993.

ϕ = geodetic latitude (positive north)

λ = geodetic longitude (measured east from the Greenwich meridian)

h = altitude normal to ellipsoid

a = ellipsoidal equatorial radius (a = 6 378.137 km for model WGS-84)

e = eccentricity of ellipsoid (e^2 = 0.00669437999 for model WGS-84)

b = ellipsoidal polar radius ($b = a\sqrt{1 - e^2}$)

2.6 Given earth-centred earth-fixed coordinates (X, Y, Z) of a point P, we can transform them into geodetic coordinates (ϕ, λ, h) through the following formulae, provided that P does not fall within 43 km of earth centre:

$$w = \sqrt{x^2 + y^2}$$

$$l = e^2/2, \quad m = (w/a)^2, \quad n = [(1 - e^2)z/b]^2$$
$$i = -(2l^2 + m + n)/2, \quad k = l^2(l^2 - m - n)$$
$$q = (m + n - 4l^2)^3/216 + mnl^2$$

$$D = \sqrt{(2q - mnl^2)mnl^2}$$
$$\beta = i/3 - \sqrt[3]{q + D} - \sqrt[3]{q - D}$$
$$t = \sqrt{\sqrt{\beta^2 - k} - (\beta + i)/2} - \operatorname{sin}(m - n)\sqrt{(\beta - i)/2}$$

$$w_1 = w/(t + l), \quad z_1 = (1 - e^2)z/(t - l)$$

$$\phi = \arctan \left[z_1 \Big/ \left((1 - e^2)w_1 \right)\right]$$
$$\lambda = 2 \arctan[(w - x)/y]$$

$$h = \operatorname{sin}(t - 1 + l)\sqrt{(w - w_1)^2 + (z - z_1)^2}$$

3. STANDARD MOLODENSKY FORMULA
(not to be used between 89° latitude and the pole)

3.1 Besides the transformation from a local geodetic datum to WGS-84 in rectangular coordinates (see Helmert's Formula), the transformation can also be performed in curvilinear (geodetic) coordinates:

$\phi_{WGS\text{-}84} = \phi_{Local} + \Delta\phi$
$\lambda_{WGS\text{-}84} = \lambda_{Local} + \Delta\lambda$
$h_{WGS\text{-}84} = h_{Local} + \Delta h$

with the Standard Molodensky Formula.

$$\Delta\phi'' = \{-\Delta X \sin\phi \cos\lambda - \Delta Y \sin\phi \sin\lambda + \Delta Z \cos\phi + \Delta a\,(ve^2 \sin\phi \cos\phi)/a + \Delta f\,[\rho\,(a/b) + v(b/a)]\sin\phi \cos\phi\} \cdot [(\rho + h)\sin 1'']^{-1}$$

$$\Delta\lambda'' = [-\Delta X \sin\lambda + \Delta Y \cos\lambda] \cdot [(v + h)\cos\phi \sin 1'']^{-1}$$

$$\Delta h = \Delta X \cos\phi \cos\lambda + \Delta Y \cos\phi \sin\lambda + \Delta Z \sin\phi - \Delta a\,(a/v) + \Delta f\,(b/a)v \sin^2\phi$$

$\Delta\phi$, $\Delta\lambda$, Δh = corrections to transform local geodetic datum to WGS-84. The units of $\Delta\phi$ and $\Delta\lambda$ are arc seconds ("); the units of Δh are metres (m)

ϕ, λ, h = geodetic coordinates (old ellipsoid) $h = H + N$ (where H is the orthometric height and N is the geoid undulation)

v = radius of curvature in the prime vertical

ρ = radius of curvature in the meridian

a, b = semi-major axis, semi-minor axis of the local geodetic datum ellipsoid ($b/a = 1 - f$)

f = flattening of ellipsoid

ΔX, ΔY, ΔZ = shift of origin

Δa, Δf = difference between the semi-major axis and the flattening of the local geodetic datum ellipsoid and the WGS-84 ellipsoid, respectively (WGS-84 minus Local)

e = eccentricity of ellipsoid ($e^2 = f(2 - f)$)

Note.— See Table B-1 for a list of reference ellipsoids and parameters and Table D-1 for a list of datum transformation parameters.

4. MULTIPLE REGRESSION EQUATION

4.1 To obtain a better fit over continental-size land areas than what could be achieved using the Standard Molodensky Formula with datum shifts ΔX, ΔY, ΔZ, the development of local geodetic datum to WGS-84 datum transformation the Multiple Regression Equation was initiated.

Note.— Local geodetic datum to WGS-84 datum transformation Multiple Regression Equations for seven major continental-size datums covering contiguous continental-size land areas with large distortion are provided in Department of Defense World Geodetic System 1984, Its Definition and Relationships with Local Geodetic Systems, *National Imagery and Mapping Agency, NIMA TR8350.2.*

$$\Delta\phi = A_0 + A_1 U + A_2 V + A_3 U^2 + A_4 UV + A_5 V^2 + K + A_{99} U^9 \mathrm{V}^9$$

$$\Delta\lambda = B_0 + B_1 U + B_2 V + B_3 U^2 + B_4 UV + B_5 V^2 + K + B_{99} U^9 V^9$$

$$\Delta h = C_0 + C_1 U + C_2 V + C_3 U^2 + C_4 UV + C_5 V^2 + K + C_{99} U^9 V^9$$

A_0, B_0, C_0	—	constant
A_i ($i = 1 \ldots 9$)	—	unknowns to be determined
B_i ($i = 1 \ldots 9$)	—	unknowns to be determined
C_i ($i = 1 \ldots 9$)	—	unknowns to be determined
$U = k\,(\phi - \phi_m)$	—	normalized geodetic latitude
$V = k\,(\lambda - \lambda_m)$	—	normalized geodetic longitude
k	—	scale factor, and degree to radian conversion
ϕ_m, λ_m	—	mean values of local geodetic datum area (in degrees)

4.2 The main advantage lies in the modelling of distortion for a better fit in geodetic applications.

5. TRANSFORMATION PARAMETERS

Table D-1 gives a list of datum transformation parameters of existing national reference frames.

Table D-1. Datum transformation parameters

Reference datum	Translations (m) ΔX	ΔY	ΔZ	Rotations (") ε_X	ε_Y	ε_Z	Scale μ (ppm)	Comments
WGS-84	·	·	·	·	·	·	·	
WGS-72	0.0	0.0	4.5	0.0	0.0	–0.554	0.22	
ED 50	–87.0	–98.0	–121.0	·	·	·	·	
ED 79	–86.0	–98.0	–119.0	·	·	·	·	
ED 87	–82.5	–91.7	–117.7	0.1338	–0.0625	–0.047	0.045	
Austria NS	595.6	87.3	473.3	4.7994	0.0671	5.7850	2.555	Via ED 87
Belgium 50	–55.0	49.0	–158.0	·	·	·	·	
Berne 1873	649.0	9.0	376.0	·	·	·	·	
CH-1903	660.1	13.1	369.2	0.8048	0.5777	0.9522	5.660	
Danish GI 1934	662.0	18.0	734.0	·	·	·	·	
Nouvelle Triangulation de France	–168.0	–60.0	320.0	·	·	·	·	Greenwich Zero Meridian
Nouvelle Triangulation de France	–168.0	–60.0	320.0	·	·	8414.03	·	Paris Zero Meridian
Potsdam	587.0	16.0	393.0	·	·	·	·	Via ED 50
GGRS 87	199.6	–75.1	–246.3	0.0202	0.0034	0.0135	–0.015	
55	–73.0	46.0	–86.0	·	·	·	·	
Ireland 65	506.0	–122.0	611.0	·	·	·	·	
Italy 1940	–133.0	–50.0	97.0	·	·	44828.40	·	Via ED 50 Rome Zero Meridian
Nouvelle Triangulation de Luxembourg	–262.0	75.0	25.0	·	·	·	·	Via ED 50
Netherlands 1921	719.0	47.0	640.0	·	·	·	·	Via ED 50
OSGB 36	375.0	–111.0	431.0	·	·	·	·	
Portugal DLX	504.1	–220.9	563.0	·	·	–0.554	0.220	Via WGS-72
Portugal 1973	–227.0	97.5	35.4	·	·	–0.554	0.220	Via WGS-72
RNB 72	–104.0	80.0	–75.0	·	·	·	·	Via ED 50
RT 90	424.3	–80.5	613.1	4.3965	–1.9866	5.1846	0.0	
NAD 27	–8.0	160.0	176.0	·	·	·	·	Mean solution
NAD 83	0.0	0.0	0.0	·	·	·	·	

Note 1.— 1" in the rotation angle is approximately equal to 31 m on the earth's surface:

1" = 6 400 000 m × 3.141593 / (180 × 3600") = 31.03 m
[1 NM = 1 852 m (so 1" = 30.48 m)]

Note 2.— 1 ppm = 10^{-6}, i.e. 1 ppm is equivalent to approximately 6.4 m on the earth's surface.

FIGURE FOR APPENDIX D

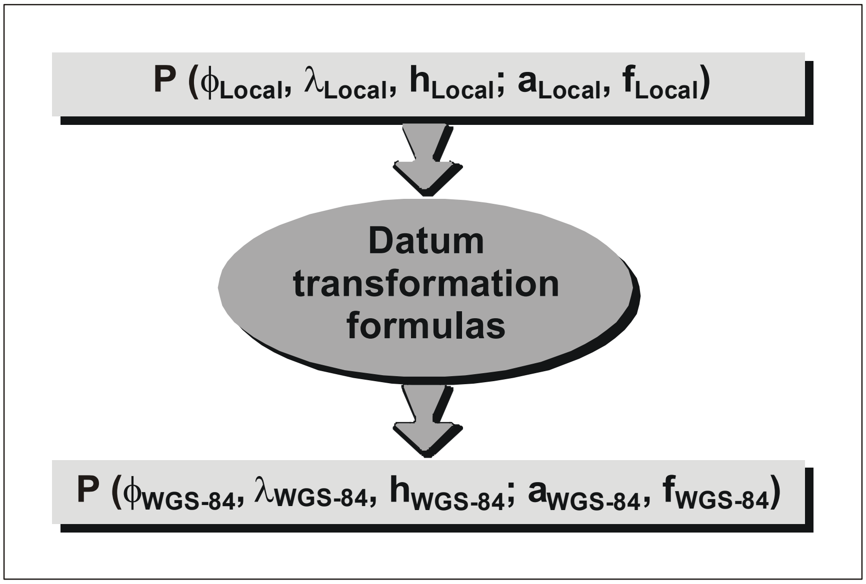


Figure D-1. General principle of a datum transformation

Appendix E

SURVEYING AND PHOTOGRAMMETRIC METHODS

1. SURVEYING

Definition of ***Surveying.*** A method of data collection by observation and measurement.

Conventional surveying techniques

1.1 WGS-84 coordinates can be obtained by collocating the point(s) with a WGS-84 (ITRF) station (measurement of relative coordinate differences).

1.2 Conventional surveying techniques determine:

a) directions and angles using a theodolite (accuracy of directions up to 0.01 mgon (0.1"). One gon is also called the grad which is 1/400 of a circumference). The technique is also called triangulation; and

b) distances by EDM (electronic distance measurement). The accuracy derived can be up to 1 mm. The technique is also called trilateration.

Total stations

1.3 Modern total stations combine a self-recording, high-precision electronic theodolite with an EDM instrument which is mounted (often) co-axially. A total station is set up over a reference point where coordinates are known and the telescope is pointed towards a target/reflector to measure distance, and horizontal and vertical angles. These are recorded automatically either for immediate display (on-line field computation capabilities) or for post-processing.

Spirit levelling

1.4 Figure E-1 shows the principle of spirit levelling, which is used to determine height differences relative to a point of known (orthometric) height. The accuracy derived can be up to ± 0.3 mm/$\sqrt{s}$ (km). Although automated instruments with complete digital data handling are available, the technique is very time-consuming, elaborate and expensive and the error propagation formula is only applicable in a local area, say, up to 50 km. Hence, the main application of spirit levelling is with respect to the local area.

1.5 Heights that are derived using a trigonometric technique (measuring zenith or height angles) can be considered as “orthometric heights” (neglecting gravity data and accuracy in decimetre level).

Advantages/disadvantages of conventional surveying

1.6 Field operations with modern total stations take a very short time and a surveyor with an assistant can typically survey up to 1 000 points per day in favourable meteorological circumstances (theodolite and EDM instruments, as well as spirit levelling instruments, are less efficient). This technique, however, has a number of drawbacks.

1.7 The observations are limited by intervisibility between theodolite and target and, to a lesser extent, by range. The technique requires a number of evenly distributed control reference stations, whose coordinates are either known in advance or determined separately. Horizontal control coordinates are normally computed by triangulation points and heights are provided by benchmarks, both obtainable from the respective national mapping organization. National survey organizations, in turn, survey these points and compute their coordinates or heights by using a series of hierarchical networks, starting at primary level and broken down successively into second order, third order, and so on. In a densely surveyed State, lower-order triangulation points are to be found, at best, at a density of, say, one every 5 km. This is not sufficient for general surveying, and additional control stations have to be established prior to a detailed survey by well-known control survey techniques such as triangulation, trilateration and traversing. GPS surveying and photogrammetric techniques are, in that respect, becoming progressively more efficient.

Heights

1.8 Processing of vertical angle and distance measurements results in so-called trigonometric height differences (= ellipsoidal height differences).

1.9 Orthometric heights of high accuracy can only be derived by measuring gravity potential differences $C = \Delta W = \int g \, dh$ (combination of gravity observations g and spirit-levelled height differences dh) and dividing it by normal gravity. In other words, to come up with high-accuracy orthometric heights, orthometric corrections have to be applied.

1.10 In a local non-mountainous region (e.g. 50 km × 50 km), the geoid variation might be < 0.1 m. Neglecting these geoid differences, the type of height becomes irrelevant (orthometric height difference = trigonometric height difference = levelled height difference).

2. PRINCIPLES OF AERO-PHOTOGRAMMETRY

2.1 A photogrammetric camera placed in an aeroplane takes overlapping photos in a strip. Overlapping photo strips form a block. The objects on the ground whose coordinates are to be determined are appropriately marked on the photos for clear identification.

2.2 After the photos are developed, precise image coordinates are determined by photogrammetric instruments. The transformation of these image coordinates into the WGS-84 coordinates of the ground stations is computed by using ground control (identifiable ground stations with WGS-84 coordinates).

Relative orientation

2.3 A stereoscopic model of the ground, using two overlapping photos, can be established. This process is called relative orientation. It uses the geometry of the photos, i.e. the central perspective, and takes account of the tilts between images which are due to displacements of the aircraft (see Figure E-2).

2.4 Analogue photo cameras are likely to be replaced in the near future by digital cameras.

Absolute orientation

2.5 The so-called absolute orientation process is computed (bundle block adjustment, see Figure E-3) by using ground control (stations with WGS-84 coordinates available or with coordinates established by GPS surveying) in each model (one pair of photos). Furthermore, for large areas, the earth's curvature has to be taken into consider-ation as well as the variation of the geoid.

Minimization of ground control

2.6 Knowing that approximately 60% to 80% of the costs of a photogrammetric project are related to the establishment of ground control, considerable savings can be achieved when determining the precise aerial camera coordinates at the moment of exposure by using kinematic differential GPS (DGPS) positioning (relative to a reference station). This requires a GPS receiver and antenna on the aircraft that are time-synchronized with the photogrammetric camera. In addition, the geometrical offset (eccentricity) between camera and GPS antenna has to be measured beforehand.

2.7 Figure E-4 shows the minimal amount of ground control required in the case where the camera position coordinates are determined by using DGPS.

Working steps

2.8 When carrying out a photogrammetric project, the following working steps are necessary:

a) The parameters of the photo flight have to be determined as a function of anticipated coordinate accuracy of the ground stations.

b) If no WGS-84 coordinates at ground stations are available, they have to be established using GPS differential surveying techniques.

d) The points to be coordinated have to be marked so that a unique identification in the aerial photos is possible.

d) Photo flights can be carried out only in clear weather conditions. For precise coordinate determination, it is necessary to fly the strips using flight guidance with approximately 50 m horizontal accuracy.

e) The image coordinates have to be measured from the established stereoscopic models in photogrammetric instruments (e.g. an "analytical plotter").

f) The final coordinates are derived computationally using a so-called photogrammetric block adjustment.

g) Verification of the photogrammetrically-derived coordinates has to be done using selected field checks (GPS or conventional surveying).

Advantages/disadvantages

2.9 The advantages of aero-photogrammetry are that:

a) a photogrammetric survey can cover a large area in one flight; and

b) the analogue photos which are taken for coordinate determination contain a lot of additional analogue information which might be useful for other tasks (e.g. interpretation).

2.10 The disadvantages may be that:

a) since flights should be carried out when the vegetation is low and the weather is clear, long waiting times may occur;

b) it might not be economical compared to other terrestrial techniques;

c) certain restrictions may arise due to flight constraints and air traffic control; and

d) the public release of photos may need approval by governmental or military organizations.

FIGURES FOR APPENDIX E

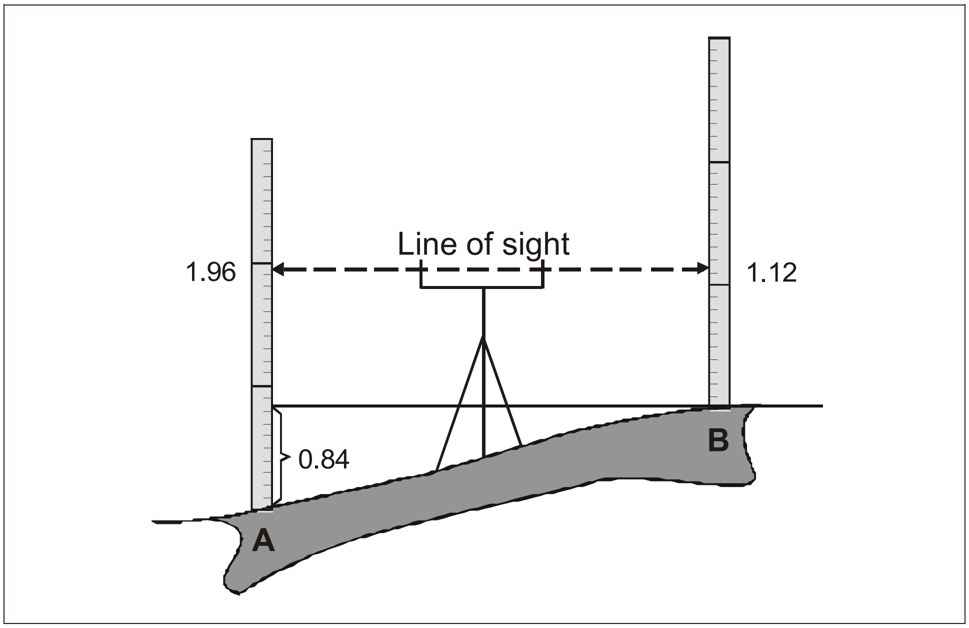


Figure E-1. Principle of spirit levelling

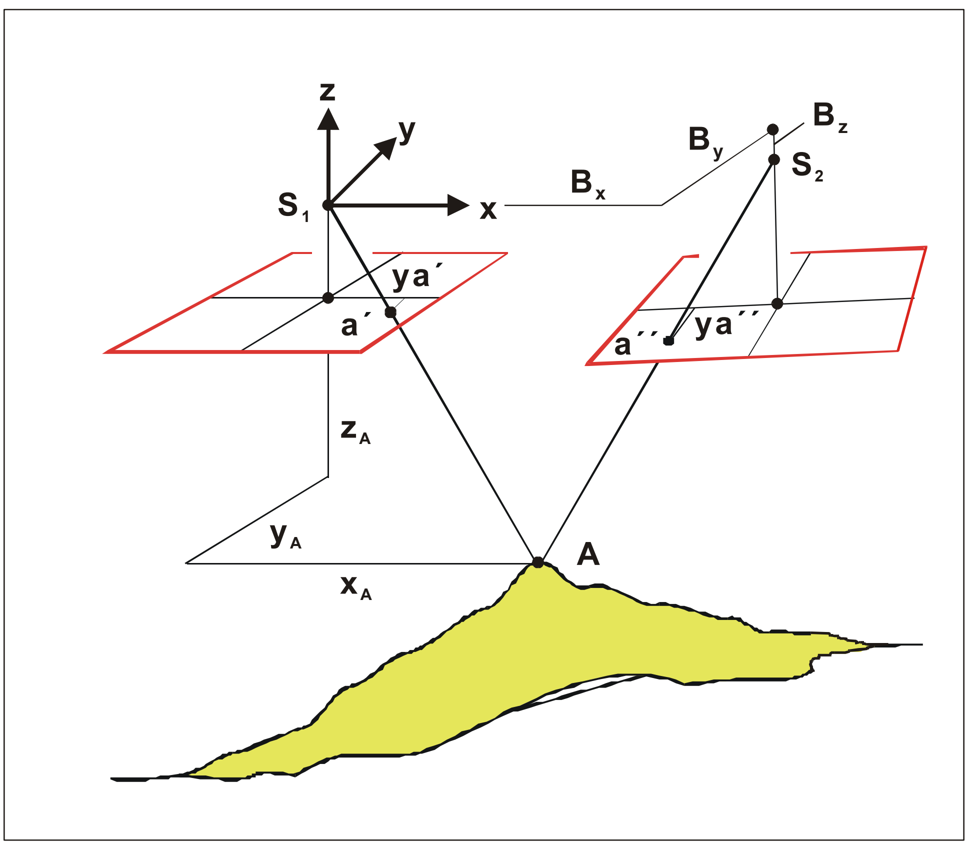


Figure E-2. Overlapping vertical photos taken by an aerial camera

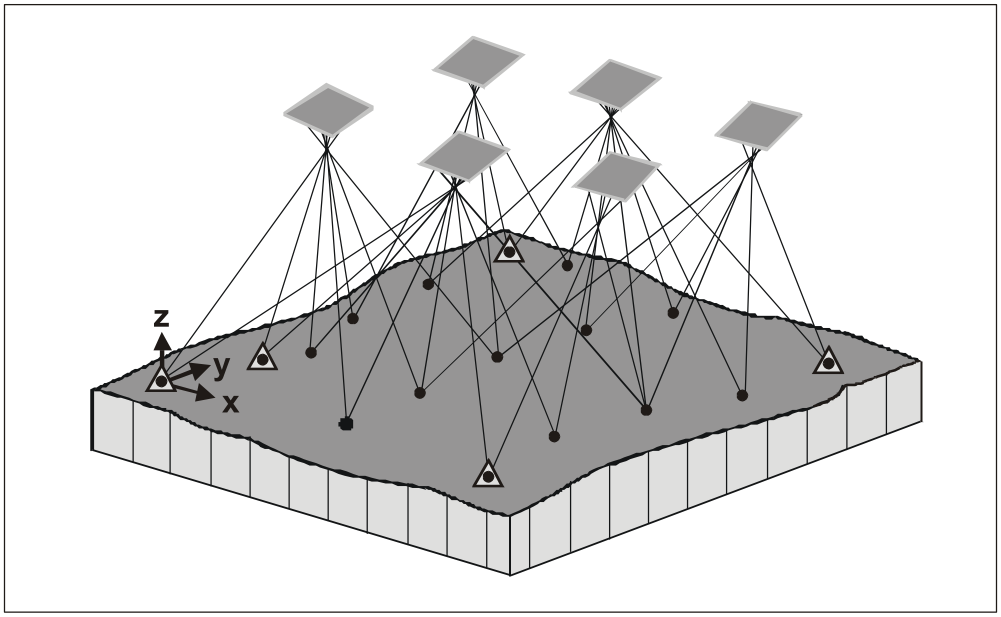


Figure E-3. Bundle block adjustment

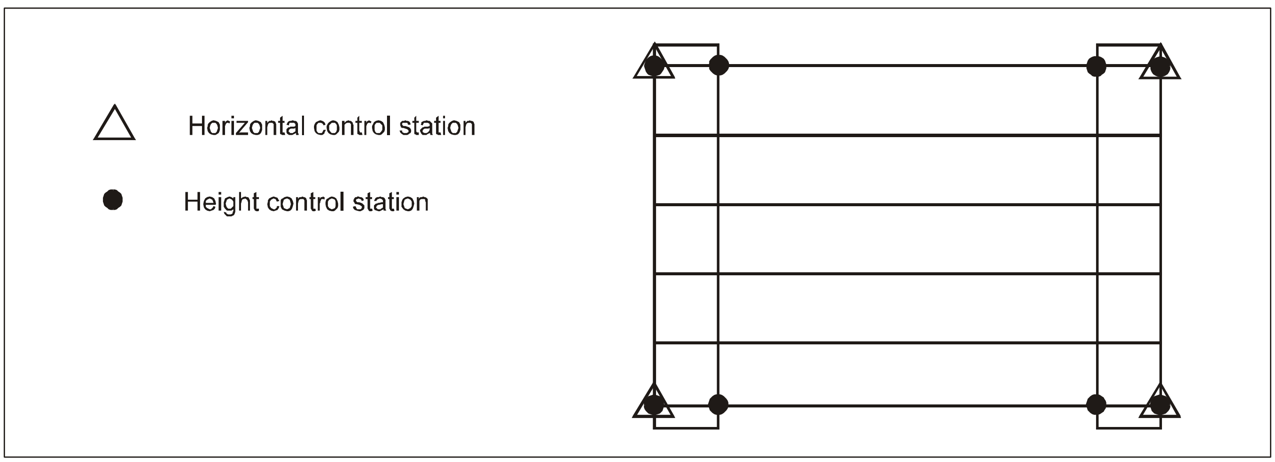


Figure E-4. Minimal amount of ground control by using DGPS

Appendix F

MAP PROJECTIONS

1. INTRODUCTION

Advances in information technology have given rise to automatic cartography and, hence, to digital mapping. A hard copy analogue map can now be digitized and transformed into a database, which can then be used for a variety of computer-aided design (CAD) applications in planning, civil engineering and geographical information systems (GIS). National surveying and mapping organizations are now well advanced in digitizing their national mapping, typically at scales from 1:1 000 for urban areas and up to 1:10 000 for rural areas.

2. GENERAL PRINCIPLE OF MAP PROJECTIONS

2.1 It is necessary to determine the functions f_1 and f_2 which map the ellipsoidal (or spherical) coordinates ϕ, λ onto a plane with rectangular coordinates X, Y.

$$X = f_1(\phi, \lambda)$$

$$Y = f_2(\phi, \lambda)$$

2.2 f_1 and f_2 can be functions of latitude, longitude, or both. Each map projection has unique equations for X and Y. In other words, there is a one-to-one correspondence between the earth and the map.

Note.— Some map projections show the same meridian twice, because the geographical poles are represented by lines instead of points or because certain parts of the earth's surface cannot be shown on the projection. These peculiarities arise from the simple fact that a sphere has a continuous surface, whereas a plane map must have a boundary.

2.3 The correspondence between points on the surface of the earth and the plane cannot be exact. In the first place, a scale change must occur. In the second place, the curved surface of the earth cannot be fitted to a plane without introducing some deformation or distortion which is equivalent to stretching or tearing the curved surface.

3. TYPES OF PROJECTIONS

3.1 Local surveys are usually computed in plane metric coordinates. These coordinates are obtained by mapping the reference ellipsoid to the plane by applying one of the well-known map projections. The surveyor mainly works in this plane coordinate system (*x* or northings, *y* or eastings). The coordinates of surveyed objects, obtained by using, for example, EDM equipment and theodolites, are most easily computed by applying plane computation formulae.

3.2 Air navigation, on the other hand, operates using geographic coordinates (latitude, longitude). Therefore the problem is to derive geographic coordinates as a function of plane coordinates. This problem can be solved by applying the inverse map projection formulae to the eastings and northings. Application of inverse map projection requires that the type of national map projection be clearly defined in mathematical terms.

3.3 Projections can be classified as mapping of the earth onto the plane of:

a) an azimuthal plane;

b) a tangent cone; or

c) a tangent cylinder.

3.4 Plane, cone and cylinder can be in a normal, transverse or oblique position attached to the earth. In addition, the surfaces of the plane, cone and cylinder can intersect the ellipsoid (or sphere) so that there are two lines of contact. These projections are called secant projections.

4. PROPERTIES OF PROJECTIONS

Equidistance

4.1 One particular scale is made equal to the principal scale throughout the map. Usually, this is the meridional scale.

Equivalence

4.2 An equal area map is one in which $a \times b = 1$ (a and b are the axes of the ellipse of distortion).

Conformal map

4.3 A conformal map is one in which $a = b$ at all points of the map.

Geodetic map projections

4.4 Geodetic map projections differ from the cartographic ones as follows:

a) The application is mainly with respect to large- or medium-scale maps.

b) The reference ellipsoid is the one used by the national surveying agency.

c) The graticule lines on the map represent geodetic coordinates (in contrast to geographic coordinates on cartographic graticules).

d) Geodetic maps should represent the results of surveying (e.g. northing, easting).

4.5 At present, mainly conformal transverse cylindrical projections are used (Gauss' coordinates).

5. SPHERICAL PROJECTION FORMULAE

Transverse cylindric Mercator

5.1 A transverse cylindric Mercator projection it is a conformal projection. The meridian is the line of zero distortion and the principal scale is preserved along this meridian. At the equator the projection is also equidistant. There is no representation of the poles. The projection may be either tangential or secant by the use of a central meridian scale factor. The X- (north) axis is towards the North Pole along the central meridian and the Y- (east) axis is directed towards the east. The origin is at a point on the central meridian which may be arbitrarily selected by the definition of a latitude of origin ϕ_0.

$$X = Rlu(\sec\theta + \tan\theta)$$

$$Y = R(\cos^{-1}(\tan\theta \cot(\lambda - \lambda_0)\text{-}\phi_0))$$

$$\theta = \sin^{-1}(\cos\phi \sin(\lambda - \lambda_0))$$

$$R = F_0\sqrt{\nu\rho}$$

ϕ_0, λ_0 = latitude of origin point,
longitude of central meridian

F_0 = central meridian scale factor
($F_0 = 0$ for tangential)

Stereographic polar

5.2 An orthomorphic zenithal (plane) projection which is tangential at the North Pole. The Cartesian axes are oriented with the X- (north) axis pointing away from the "central" meridian, and the Y- (east) axis pointing towards 90° east of the central meridian.

$$X = \frac{-2R\cos\phi\cos(\lambda - \lambda_0)}{1 + \sin\phi}$$

$$Y = \frac{2R\cos\phi\cos(\lambda - \lambda_0)}{1 + \sin\phi}$$

R = radius of the sphere, as given by $R = \sqrt{\nu\rho}$

λ_0 = longitude of central meridian

Stereographic oblique

5.3 Similar to the Stereographic polar with the exception that the plane is tangential to the sphere at any point. The X- (north) axis is directed towards the North Pole and the Y- (east) axis to the east.

$$X = 2R\,\frac{\sin\phi\cos\phi_0 - \cos\phi\sin\phi_0\cos(\lambda - \lambda_0)}{1 + \sin\phi\sin\phi_0 + \cos\phi\cos\phi_0\cos(\lambda - \lambda_0)}$$

$$Y = 2R\,\frac{\cos\phi\sin(\lambda - \lambda_0)}{1 + \sin\phi\sin\phi_0 + \cos\phi\cos\phi_0\cos(\lambda - \lambda_0)}$$

ϕ_0, λ_0 = latitude and longitude of tangential point

Lambert conical, one standard parallel

5.4 An orthomorphic conical normal projection with the cone tangential to the sphere along one standard parallel of latitude. The Cartesian axes are oriented with the X- (north) axis towards the North Pole and the Y- (east) axis to the east.

$X = r_0 - r \cos \theta$

$Y = r \sin \theta$

$\theta = (\lambda - \lambda_0)\sin \phi_0$

$r_0 = R \cot \phi_0$

$$r = r_0\left(\frac{\tan(45 - 0.5\phi)}{\tan(45 - 0.5\phi_0)}\right)^{\sin\phi_0}$$

$$R = \sqrt{v\rho}$$

ϕ_0, λ_0= latitude of standard parallel, longitude of central meridian

Lambert conical, two standard parallel

5.5 Similar to the tangential case but with a secant cone which cuts the sphere at two standard parallels, ϕ_1 and ϕ_2. The origin of the Cartesian coordinates is at an implied mid-latitude ϕ_0.

$X = r_0 - r \cos \theta$

$Y = r \sin \theta$

$\theta = (\lambda - \lambda_0)\sin \phi_0$

$$l_0 = \frac{R \cos\phi_1}{\sin\phi_0}\left(\frac{\tan(45 - 0.5\phi_0)}{\tan(45 - 0.5\phi_1)}\right)^{\sin\phi_0}$$

$$r = \frac{R \cos\phi_1}{\sin\phi_0}\left(\frac{\tan(45 - 0.5\phi_0)}{\tan(45 - 0.5\phi_1)}\right)^{\sin\phi_0}$$

$$\sin \phi_0 = \frac{lu \cos \phi_1 - lu \cos \phi_2}{lu \tan(45 - 0.5\phi_1) - lu \tan(45 - 0.5\phi_2)}$$

6. ELLIPSOIDAL PROJECTION FORMULAE

Note.— For the ellipsoidal formulae, the descriptions and definitions of Cartesian axes are the same as for the spherical formula.

6.1 Stereographic polar

$X = -r \cos \theta$

$Y = r \sin \theta$

$\theta = \lambda - \lambda_0$

λ_0 = longitude of central meridian

$$r_0 = 2a(1+e)^{\frac{-1}{2(1-e)}}(1-e)^{\frac{-1}{2(1-e)}}\left(\frac{\cos\phi}{1+\sin\phi}\right)\left(\frac{1+e\sin\phi}{1-e\sin\phi}\right)^{1/2e}$$

a = semi-major axis of ellipsoid

e = eccentricity of ellipsoid

6.2 Stereographic oblique

$$X = 2v_0\left(\frac{1+e}{1-e}\right)^{-1/2}\left(\frac{\sin\phi \ \cos\phi_0 - \cos\phi \ \sin\phi_0 \ \cos(\lambda-\lambda_0)}{1+\sin \ h}\right)\left(\frac{1+e \ \sin \ h}{1-e \ \sin \ h}\right)^{-1/2}$$

$$Y = 2v_0\left(\frac{1+e}{1-e}\right)^{-1/2}\left(\frac{\cos\phi \ \sin(\lambda-\lambda_0)}{1+\sin \ h}\right)\left(\frac{1+e \ \sin \ h}{1-e \ \sin \ h}\right)^{-1/2}$$

$\sin h = \sin \phi \sin \phi_0 + \cos \phi \cos \phi_0 \cos(\lambda - \lambda_0)$

$$v_0 = \frac{a}{(1-e^2 \ \sin^2 \ \phi_0)^{1/2}}$$

f_0, l_0 = latitude of tangential pole, longitude of tangential pole

6.3 Lambert conical, one standard parallel

$X = r_0 - r \cos \theta$

$Y = r \sin \theta$

$\theta = (\lambda - \lambda_0)\sin \phi_0$

$r_0 = v_0 \cot \phi_0$

$$r = r_0\left(\frac{\tan(45-0.5\phi)\left(\frac{1+e\ \sin\ \phi}{1-e\ \sin\ \phi}\right)^{1/2e}}{\tan(45-0.5\phi_0)\left(\frac{1+e\ \sin\ \phi_0}{1-e\ \sin\ \phi_0}\right)^{1/2e}}\right)^{\sin\phi_0}$$

$$v_0 = \frac{a}{(1-e^2\ \sin^2\ \phi_0)^{1/2}}$$

f_0, l_0 = latitude of standard parallel, longitude of the central meridian

6.4 Lambert conical, two standard parallel

$$X = r_0 - r\cos\theta$$

$$Y = r\sin\theta$$

$$\theta = (\lambda-\lambda_0)\sin\phi_0$$

$$r_0 = v_2\frac{\cos\phi_2}{\sin\phi_2}\left(\frac{\tan(45-0.5\phi_0)\left(\frac{1+e\ \sin\ \phi_0}{1-e\ \sin\ \phi_0}\right)^{1/2e}}{\tan(45-0.5\phi_2)\left(\frac{1+e\ \sin\ \phi_2}{1-e\ \sin\ \phi_2}\right)^{1/2e}}\right)^{\sin\phi_0}$$

$$r = v_2\frac{\cos\phi_2}{\sin\phi_0}\left(\frac{\tan(45-0.5\phi)\left(\frac{1+e\ \sin\ \phi}{1-e\ \sin\ \phi}\right)^{1/2e}}{\tan(45-0.5\phi_0)\left(\frac{1+e\ \sin\ \phi_0}{1-e\ \sin\ \phi_0}\right)^{1/2e}}\right)^{\sin\phi_0}$$

$$\sin\phi_0\frac{luv_1\ \cos\phi_1 - luv_2\ \cos\phi_2}{lu\left(\tan(45-0.5\phi_1)\left(\frac{1+e\ \sin\ \phi_1}{1-\ e\ \sin\ \phi_1}\right)^{1/2e}\right) - lu\left(\tan(45-0.5\phi_2)\left(\frac{1+e\ \sin\ \phi_2}{1-\ e\ \sin\ \phi_2}\right)^{1/2e}\right)}$$

$$v_1 = \frac{a}{(1-e^2\ \sin^2\ \phi_1)^{1/2}}$$

$$v_2 = \frac{a}{(1-e^2\ \sin^2\ \phi_1)^{1/2}}$$

6.5 Transverse cylindric Mercator

6.5.1 The transverse Mercator projection from the ellipsoidal surface is usually expressed as a number of series expansions.

$$X = M + (\lambda-\lambda_0)^2 A + (\lambda-\lambda_0)^4 B + (\lambda-\lambda_0)^6 C$$

$$Y = (\lambda-\lambda_0) D + (\lambda-\lambda_0)^3 E + (\lambda-\lambda_0)^5 F$$

$$M = b(1 + n + \frac{5}{4}n^2 + \frac{5}{4}n^2)(\phi-\phi_0)$$

$$-(3n+3n^2+\frac{21}{8}n^3)\sin(\phi-\phi_0)\cos(\phi+\phi_0)$$

$$+(\frac{15}{8}n^2+\frac{15}{8}n^3)\sin 2(\phi-\phi_0)\cos 2(\phi+\phi_0)$$

$$-\frac{25}{24}n^3\sin 3(\phi-\phi_0)\cos 3(\phi+\phi_0)$$

$$A = \frac{v}{2}\sin\phi\cos\phi$$

$$B = \frac{v}{24}\sin\phi\cos^3\phi(5-\tan^2\phi+9\eta^2)$$

$$C = \frac{v}{720}\sin\phi\cos^5\phi(61-58\tan^3\phi+\tan^4\phi)$$

$$D = v\cos\phi$$

$$E = \frac{v}{6}\cos^3\phi(\frac{v}{\rho}-\tan^2\phi)$$

$$F = \frac{v}{120}\cos^5\phi(5-18\tan^2\phi+\tan^4\phi+14\eta^2-58\tan^2\phi\eta^2)$$

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

$$\rho = \frac{a(1-e^2)}{(1-e^2\sin\phi)^{3/2}}$$

$$\eta = \frac{a-b}{a+b}$$

$$b = a(1-f)$$

$$\eta^2 = \frac{v}{\rho} - 1$$

6.5.2 The central meridian scale factor F_0 is applied by multiplying the semi-major axis a by F_0 before calculating any other quantities.

7. GAUSS-KRÜGER PROJECTION

The Gauss-Krüger projection is identical to the conformal Mercator projection but the position of the cylinder is transverse. The reference surface is the Bessel ellipsoid and the central meridian is equidistant. The *y*-coordinates (eastings) are enlarged due to the convergence of the *x*-axes (northings). In order to keep this distortion in reasonable limits, zones of 3° width (in longitude) are established with the central meridian lying in the middle of each zone. The meridional strip systems have the following northings and eastings:

Northing: Distance from the equator.

Easting: In order to avoid negative signs, each central meridian has the constant y = 500 000 m. The first two digits represent the zone number which is the longitude of the central meridian divided by three.

Example: The tower of the town hall of Berlin has the geographical coordinates:

λ = 13°24"36.01" ϕ = 52°31"11.65"

The corresponding Gauss-Krüger coordinates with respect to the central meridian of λ = 12° are:

E = 45 95 696.00 m N = 5821529.20 m.

8. UNIVERSAL TRANSVERSE MERCATOR (UTM) SYSTEM

The Universal Transverse Mercator (UTM) system, often used for military maps, comprises the following features:

a) The projection is the Gauss-Krüger version of the Transverse Mercator intended to provide world coverage between the latitudes 84°N and 80°S.

b) The reference ellipsoid is the International 1924; the unit of measure is the International Metre.

c) Each zone is 6° of longitude in width. The first zone has its western edge on the meridian 180° and the zones proceed eastwards to zone 60 which has its eastern edge at 180° longitude. The central meridian of each zone is therefore: 177° in zone 1; 171° in zone 2; and 168° in zone 3, etc.

d) The origin of each zone is the point on the equator where it is intersected by the central meridian of the zone. Each zone extends as far polewards as 84° N and 80° S (initially these limits were set at 80° N and 80° S).

e) The eastings of the origin of each zone are assigned the value of 500 000 m.

f) The scale factor on the central meridian is 0.9996.

g) The UTM employs five different figures for specific areas.

Appendix G

SAMPLE QUESTIONNAIRE

Survey inventory related to WGS-84 implementation

Notes on the completion of Parts I and II:

1. *Part I should be completed by national aviation administrations.*

2. *Part I, 25, items i to ix, should be completed for each navigation aid.*

3. *Part II should be distributed by national aviation administrations to aerodrome/heliport authorities for completion — one questionnaire for each aerodrome/heliport. The cover page is to be completed by national administrations.*

4. *Initially, Part II should be completed for aerodromes/heliports that have established instrument approach procedure(s) for the runway(s).*

5. *For aerodromes/heliports where visual approach procedures only are established, the requirement is for information on the aerodrome/heliport reference point (ARP).*

6. *Parts I and II have been designed so that the responses can be scanned automatically into a digital format. The majority of the questions are in a multiple-choice format. The scanning and formatting of the data will permit analysis using a PC-based tool. This PC programme is available to States from ICAO.*

7. *Blank templates of the questionnaires are available from ICAO. These may be used by national administrations to insert text in their national language. It is important that the format and layout be maintained so that translated questionnaires may be scanned and analysed by the PC programme referred to in 6 above.*

Part I — En-route navigation aids

State

Civil Aviation Administration

Address

Contact name

Telephone

Facsimile

Navigation Aid Survey Inventory

Questionnaire

Part I — En-route navigation aids

(DME, VOR, DME/VOR, NDB, VORTAC and TACAN)

1. By which organization is coordination of navigation aids performed?

[ ] National aviation administration's internal survey unit
[ ] Aerodrome/heliport staff surveyors
[ ] National mapping agency
[ ] Local government survey unit
[ ] Military survey department
[ ] Private survey contractor
[ ] Other — please specify |__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|

2. Is a written specification used for the method of measurement?

[ ] Yes [ ] No

3. Which facilities are surveyed?

DME	[ ]	Yes	[ ]	No
VOR	[ ]	Yes	[ ]	No
VOR/DME	[ ]	Yes	[ ]	No
NDB	[ ]	Yes	[ ]	No
VORTAC	[ ]	Yes	[ ]	No
TACAN	[ ]	Yes	[ ]	No

4. Is coordination done by instrument survey or by graphical methods?

DME	[ ]	Instrument survey	[ ]	Graphical method
VOR	[ ]	Instrument survey	[ ]	Graphical method
VOR/DME	[ ]	Instrument survey	[ ]	Graphical method
NDB	[ ]	Instrument survey	[ ]	Graphical method
VORTAC	[ ]	Instrument survey	[ ]	Graphical method
TACAN	[ ]	Instrument survey	[ ]	Graphical method

5. If coordinates are obtained graphically from map products, what map scale (or nearest equivalent) is generally used?

	DME	VOR	DME/VOR	NDB	VORTAC	TACAN
>1/5 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/5 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/10 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/20 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/25 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/50 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/100 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/250 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1/250 000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]

6. To what accuracy are the coordinates determined?

Note.— This may differ from the resolution quoted in the AIP.

	DME	VOR	DME/VOR	NDB	VORTAC	TACAN
>1 NM	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1 NM	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
0.1 NM	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
100 m	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
10 m	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1 m	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]

INFRASTRUCTURE

7. If coordinates are extracted from a map, is it known on which datum the map is based?

[ ] Yes [ ] No

8. Is the information in 7 above recorded as part of the survey?

[ ] Yes [ ] No

9. If instrument surveys are performed,

a) Is a record made of the reference frame used?

[ ] Yes [ ] No

b) Are permanent survey stations established as part of the survey?

[ ] Yes [ ] No

QUALITY CONTROL

10. Is the determination of the geographical coordinates of navigation aids covered by a formal system of quality assurance, such as ISO 9000 or equivalent?

[ ] Yes — please specify | | | | | | | | | | | | |
[ ] No

11. What classification of staff is used for coordinating en-route navaids?

[ ] Professional surveyors
[ ] Qualified cartographers or draughtsmen
[ ] Qualified technicians
[ ] Junior grade staff
[ ] Untrained staff
[ ] Not known

12. Is specific training given for the particular task of surveying navigation aids?

[ ] Yes [ ] No

13. Are field inspections undertaken to verify the location of the navigation aids and, if yes, are they part of an ongoing programme for inspection?

[ ] Yes [ ] No

[ ] Yes [ ] No

14. Are such inspections, or similar inspections, part of an ongoing programme for inspection or calibration?

[ ] Yes [ ] No

15. Where coordinates are supplied by other government agencies, is any further form of checking performed?

[ ] Yes [ ] No

RECORDS AND ARCHIVES

16. Are comprehensive records kept on positioning and coordinate data ?

[ ] Yes [ ] No

17. Are such records free of inconsistencies?

[ ] Yes [ ] No

18. Is it possible to trace the data and the method of the survey/coordination of individual navaids?

[ ] Yes [ ] No

19. Are the survey records held centrally and, if yes, are they easily accessible?

[ ] Yes [ ] No

[ ] Yes [ ] No

20. Are the survey records held on computer?

[ ] Yes [ ] No

21. Are the survey records subject to regular maintenance?

[ ] Yes [ ] No

22. In the case of collocated navigation aids (VOR/DME), is it known to which facility the published coordinates relate?

[ ] Yes [ ] No

23. Is the physical separation of such pairs of facilities known?

[ ] Yes [ ] No

24. Where central records of precise coordinates of navigation aids are kept, are the published AIP coordinates checked for consistency?

[ ] Yes [ ] No

CONFIRMATION OF AIP ENTRY

25. Please indicate in Column A the number of navaids for which coordinates are published. In Column B state the number of navaids for which the coordinates are determined by the national civil aviation administration itself.

AID	Column A	Column B
DME		
VOR		
VOR/DME		
NDB		
VORTAC		
TACAN		

The following information is required for each DME, VOR, DME/VOR, TACAN and VORTAC. The information is not required for NDBs.

(i) Identifier

(ii) Type of aid

[ ] DME [ ] VOR [ ] DME/VOR
[ ] TACAN [ ] VORTAC

(iii) Is there any documentary record of the date of survey (i.e. date on which coordinates were determined)?

[ ] Yes [ ] No

(iv) Method of survey, please specify.

(v) To which geodetic datum do the coordinates refer?

[ ] WGS-84 [ ] National Mapping Datum
[ ] ED 50 [ ] Not known

(vi) If the answer to (v) was national mapping datum, please state its name.

(vii) Are original survey observations available?

[ ] Yes [ ] No

(viii) In the case of VOR, TACAN and VORTAC, please state, if available, the value of the SITE DECLINATION (VARIATION) set at the last calibration.

DEGREES.XX [ ] East [ ] West

Date of last calibration

(ix) For collocated aids (e.g. VOR/DME), do the coordinates relate to the distance measuring element?

[ ] Yes [ ] No

Part II — Aerodrome/heliport points

Aerodrome/heliport ICAO location indicator	
Aerodrome/heliport	
Address	
Contact name	
Telephone	
Facsimile	

Navigation Aid Survey Inventory

Questionnaire

Part II — Aerodrome/heliport points

This section to be completed by national administrations.

First name	
Family name	
Position	
Address	
Telephone	
Facsimile	

This section to be completed by the technical officer responsible for providing survey details.

First name

Family name

Position

Address

Telephone

Facsimile

SURVEY CONTROL NETWORK

1. Is there an existing survey control network at the aerodrome/heliport?

   [ ] Yes   [ ] No   [ ] Not known

2. If yes, who installed and surveyed the network?

   [ ] National aviation administration's internal survey unit
   [ ] Aerodrome/heliport staff surveyor
   [ ] National mapping agency
   [ ] National geodetic agency
   [ ] Local government survey unit
   [ ] Military survey department
   [ ] Private survey contractor
   [ ] Other — specify |__|__|__|__|__|__|__|__|__|__|__|__|__|__|

3. State the name of the survey organization. (If more than one survey organization has been involved, state the name of the organization having done the most work.)

   |__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|

4. State the age of the aerodrome/heliport survey control network, in years.

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

5. To which geodetic datum or control frame is the network referenced?

   [ ] WGS-84   [ ] National Mapping Datum
   [ ] ED 50   [ ] Not known

6. To which order of geodetic network is the aerodrome/heliport survey network connected?

   [ ] Primary   [ ] Secondary   [ ] Tertiary

7. Name of geodetic network.

   |__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|__|

8. Method of connection.

   [ ] Static GPS   [ ] EDM Traverse
   [ ] Triangulation   [ ] Not known

9. How many stations are in place at the aerodrome/heliport?

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

10. What is the relative survey accuracy of the aerodrome/heliport survey control network?

[ ] < 1 metre [ ] < 0.1 metre [ ] < 0.02 metre

11. Types of survey control markers (monuments) used on network.

Concrete:

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

Earth anchor:

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

Surface mark (rivet, stud, metal plate, etc.):

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

12. Are survey station descriptions available?

[ ] Yes [ ] No [ ] Not known

13. Was the survey performed in accordance with the guidance in this manual?

[ ] Yes [ ] No [ ] Not known

14. Is the survey work performed in accordance with a quality assurance scheme (such as ISO 9000)?

[ ] Yes — specify |__|__|__|__|__|__|__|__|__|__| [ ] No scheme

15. Is a survey report available?

[ ] Yes [ ] No [ ] Not known

16. Has a record been kept of the original survey observations?

[ ] Yes [ ] No [ ] Partial

17. Were independent checks made on the survey?

[ ] Yes [ ] No [ ] Not known

18. Which organization made the checks?

19. Were records kept of the checks?

[ ] Yes [ ] No [ ] Not known

AERODROME OBSTACLES

20. Which organization performed the obstacle survey?

[ ] National aviation administration's internal survey unit
[ ] Aerodrome/heliport staff surveyor
[ ] National mapping agency
[ ] National geodetic agency
[ ] Local government survey unit
[ ] Military survey department
[ ] Private survey contractor
[ ] Other — specify

21. State the name of survey organization. (If more than one survey organization has been involved, state the name of the organization having done the most work.)

22. State how frequently, in years, the aerodrome/heliport obstacle survey is performed?

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

23. How long ago, in years, was the last aerodrome/heliport obstacle survey carried out?

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

24. To which geodetic datum is the aerodrome/heliport obstacle survey referenced?

[ ] National datum [ ] Local datum

25. Are the obstacle coordinates obtained by original instrument survey or by graphical means from existing mapping?

[ ] Instrument survey [ ] Existing mapping

26. Was the survey performed in accordance with any declared specifications?

[ ] Yes [ ] No [ ] Not known

27. Do full survey records for the aerodrome/heliport obstacles exist?

[ ] Yes [ ] No [ ] Not known

28. Is the survey work performed in accordance with a quality assurance scheme (such as ISO 9000)?

[ ] Yes — specify [ ] No scheme

29. Is the survey report available?

[ ] Yes [ ] No [ ] Not known

30. Has a record been kept of the original survey observations?

[ ] Yes [ ] No [ ] Not known

31. Were independent checks made on the survey?

[ ] Yes [ ] No [ ] Not known

32. Which organization made the checks?

AERODROME/HELIPORT NAVIGATION ELEMENTS

33. Which organization performed the aerodrome/heliport navigation elements survey?

[ ] National aviation administration's internal survey unit
[ ] Aerodrome/heliport staff surveyor
[ ] National mapping agency
[ ] National geodetic agency
[ ] Local government survey unit
[ ] Military survey department
[ ] Private survey contractor
[ ] Other — specify

34. State the name of survey organization. (If more than one survey organization has been involved, state the name of the organization having done the most work.)

RUNWAY THRESHOLDS

35. How many runway thresholds are on the aerodrome/heliport?

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

36. How many runways with precision approach facilities are on the aerodrome/heliport?

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

37. How many thresholds have been surveyed?

[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1	1	2	3	4	5	6	7	8	9	10	>10

38. To which accuracy have these thresholds been surveyed?

[ ] < 10 metres [ ] < 3 metres [ ] < 1 metre [ ] < 0.1 metre

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