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WGS 84 IMPLEMENTATION MANUAL

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FOREWORD

1. 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 3.2/1 - 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. "

2. The ICAO Council at the ninth meeting of its 141st Session, on 28 February 1994, adopted amendment 28 to Annex 15. Consequential amendments to Annexes 4, 11 and 14, Volume I and II will be adopted by the Council in due course. The Standards and Recommended Practices (SARPS) in Annexes 11 and 14, Volumes I and II govern the determination and reporting of the geographical coordinates in terms of WGS 84 geodetic reference system. Annexes 4 and 15 SARPS govern the publication of information in graphic and textual form. The States' aeronautical information service department will publish in Aeronautical Information Publications (AIP), on charts and in electronic data base where applicable, geographical coordinate values based on WGS 84 which are supplied by the other State aeronautical services, i.e. the air traffic services department and the aerodrome engineering department.

3. The purpose of this manual is to furnish guidance in the provision of geographical coordinates referenced to the WGS 84 datum in order to assist States in the uniform implementation of the SARPS on WGS 84.

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LIST OF ACRONYMS

A-S	Anti-Spoofing
AIP	Aeronautical Information Publications
AIS	Aeronautical Information Service
AOC	Airport Obstruction Chart
AOC	Auxiliary Output Chip
ARINC	Aeronautical Radio INC provides secretariat and technical staff for various airline committees. The ARINC 424 is a format for the transfer of data from one data base to another.
ARP	Aerodrome Reference Point
ATC	Air Traffic Control
ATS	Air Traffic Services
BIH	Bureau International de l´Heure
C/A-Code	Coarse/Acquisition-Code
CAD	Computer Aided Design
CERCO	Comité Européen des Responsables de la Cartographie Officielle
CIO	Conventional International Origin
CTA	Control Area
CTP	Conventional Terrestrial Pole
CTZ	Control Zone
CTS	Conventional Terrestrial System
DGPS	Differential GPS
DME	Distance Measurement Equipment
E-AIP	Electronic AIP
ECAC	European Civil Aviation Conference
ECEF	Earth Centred, Earth Fixed
EDM	Electronic Distance Measurement
EGM	Earth Gravity Field Model
ETRF	European Terrestrial Reference Frame
EUREF	European Geodetic Reference System
FAA	Federal Aviation Administration (USA)
FACF	Final Approach Course Fix

FAF	Final Approach Fix
FANS	Future Air Navigation System
FIR	Flight Information Region
FMS	Flight Management System
GIS	Geographical Information Systems
GLONASS	Global Navigation Satellite System
GNSS	Global Navigation Satellite System
GP	Generating Polynomial
GPS	Global Positioning System
GRS 80	Geodetic Reference System 1980
IAG	International Association of Geodesy
ICAO	International Civil Aviation Organization
IERS	International Earth Rotation Service
INS	Inertial Navigation System
IRM	IERS Reference Meridian
IRP	IERS Reference Pole
ISO	International Organization for Standardization
ITRF	International Terrestrial Reference Frame
ITRS	International Terrestrial Reference System
LAN	Local Area Network
MAPt	Missed Approach Point
MCS	Master Control Station
MSL	Mean Sea Level
NAVSTAR	NAVigation System with Time And Ranging
NDB	Nondirectional Beacon
NNSS	Navy Navigation Satellite System
OCS	Operational Control System
P-Code	Precision-Code
PDOP	Position Dilution of Precision
PPS	Precise Positioning Service
PRN	Pseudo-Random Noise
PRNAV	Precision Area Navigation
RNAV	Area Navigation
RNP	Required Navigation Performance

RTCA	Radio Technical Commission for Aeronautics
S/A	Selective Availability
SARPS	Standards and Recommended Practices
SAVVAN	Système Automatique de Vérification en Vol des Aides a la Navigation, i.e. Automatic In-flight Navigation Aids Checking System
SID	Standard Instrument Departure
SLR	Satellite Laser Ranging
SPS	Standard Positioning Service
STAR	Standard Instrument Arrival
UAC	Maastricht Upper Airspace Centre
UDDF	Universal Data Delivery Format
UIR	Upper Information Region
UTM	Universal Transverse Mercator
VLBI	Very Long Base-Line Interferometry
VOR	Very High Frequency Omnidirectional Range
WGS 84	World Geodetic System 1984

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CHAPTER 1

INTRODUCTION

1.1 Effects of using differing coordinate reference systems in aviation

Datum problems in air navigation were first encountered at EUROCONTROL in the early 1970's during the development of multi-radar tracking systems for the 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 (ATC). Discrepancies in the radar tracks were found to be the result of incompatible coordinates.

In the mid-1970's, 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 countries. Once more, the errors could only be attributed to incompatibility of the coordinates of ground aids.

Main Reason for this Effect

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

Thus - the main source of systematic errors is the non-use of a common geodetic reference datum for quoting the radar positions and its solution is to derive the radar positions in a common system.

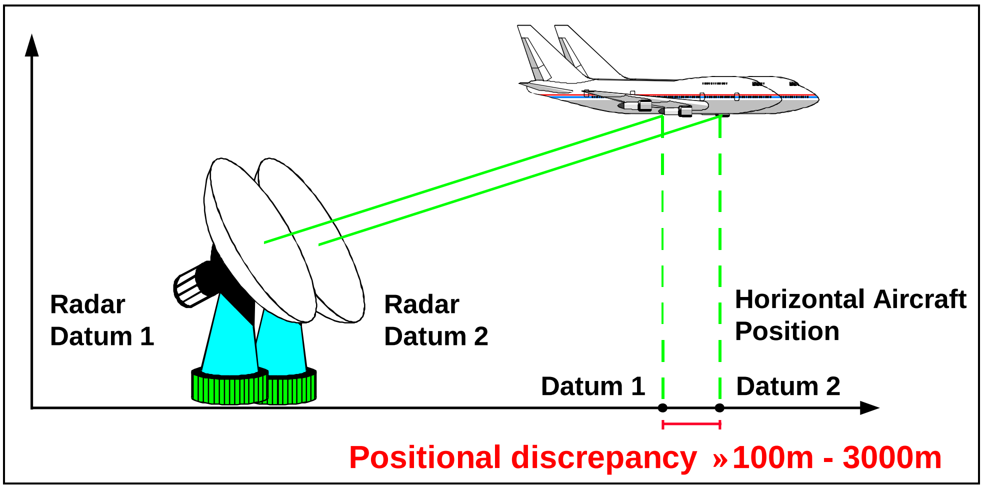

Fig. 1-1. *Datum problem in air navigation*

Datum, geodetic reference frame, ellipsoid and the geoid are explained in Appendix B

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 last century 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. Different countries developed their own geodetic datums which usually differed from those of adjacent countries. As distance requirements increased beyond national boundaries, new requirements arose from datums on at least continental scale.

If we look at the current situation, we have to acknowledge 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 the published co-ordinates do not affect the track flown by the aircraft.

More on RNAV and RNP in Chapter 2

Nevertheless, this will change dramatically either in the approach phase or landing, or where reduced lateral aircraft separation is implemented, i.e. the future Area Navigation (RNAV) or RNP (Required Navigation Performance) systems with higher accuracy and integrity requirements. Therefore, these discrepancies will no longer be tolerable and demand the introduction of a common geodetic reference system.

More on the definition and realization of WGS 84 in Chapter 3

The United States Department of Defense (World Geodetic System Committee) has 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 has 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

As already outlined the datum discrepancies between one geodetic reference frame and another depend upon:

- Order of magnitude of the three origin shifts
- Magnitude of the three axial rotations
- Scale factor value
- Shape of the reference ellipsoid (if working in geographical coordinates).

For historical reasons almost all countries already have a national reference frame with a specific set of datum parameters. The datum discrepancies range from metres to kilometres.

See Chapter 1.1 and Tab. E-1

Fig. 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 Fig. 1-2 it can be deduced that the differences in position of points with respect to different national geodetic datums and WGS-72 can attain an order of a few hundreds of metres for a particular country.

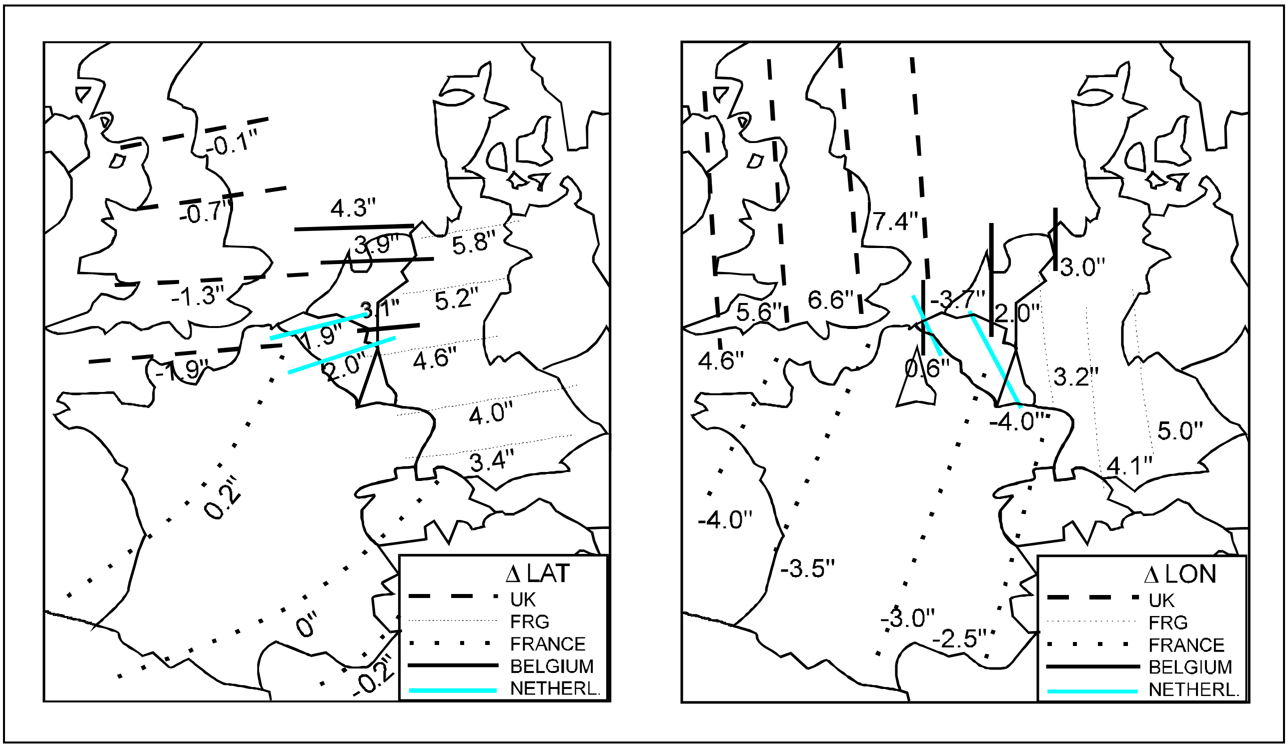

Fig. 1-2. *DLAT, DLON between local and WGS-72 (‘’)*

1.3 Navigational implications

Geographical coordinates used in the aviation environment today are generally of two types, i.e. 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 documents 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.

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 and in Europe to the European Datum (ED). Each of these datums uses a different mathematical model that "best fits" or provides the best representations of the Earth's shape in that specific geographic region. Even though aviation documents seldom publicise 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 countries which have already converted to the use of an earth centred datum, none of the datum centres coincide with the centre of gravity of the Earth.

For NAD, ED see Tab. E-1 of Appendix E

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 aligned with local coordinates before take-off, it is 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 .

Coordinates derived by the Global Navigation Satellite System (GNSS) airborne system from signals received from satellites will be Earth-centred because the GNSS satellites operate with an Earth-centred reference model, 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 Approved solution to the problem

The solution to the problems in chapter 1.1 to 1.3 is the implementation of WGS 84 as a common geodetic reference frame.

The first step in the implementation of any coordinate transformation proposal will be always an inventory. In order to make an assessment of the current quality of the published geographical coordinates of aeronautical facilities required for air navigation, it is necessary to review all existing data.

A survey inventory questionnaire designed for this purpose has already been developed by EUROCONTROL. Information provided by the use of such a questionnaire will allow the identification of those items for which a field survey is required in order to verify positions. This will allow accurate estimates to be made of the survey work that will need to be carried out.

The questionnaire is provided for reference in Appendix H

Analysis of the questionnaire data will identify the navigation aids and aerodrome points and facilities which need to be re-surveyed. Where coordinates are known to the required accuracy and integrity it will allow direct transformation to the WGS 84 geodetic reference frame by mathematical means.

In principle there are two approaches which can be used as stand-alone or combined methods to transform a survey given in adequately precise national coordinates to WGS 84 (or to a reference frame compatible with WGS 84 being earth centred and having a sufficient accuracy. Suitable reference frames would be the International Terrestrial Reference Frame (ITRF) and a local version of this called the European Terrestrial Reference Frame (ETRF)):

More on transformations and how to get WGS 84 coordinates in Chapter 4.
More on ETRF and ITRF in Appendices C and D

- Surveying at least three control stations (covering the area under consideration) to obtain WGS 84 coordinates, and determining the datum parameters between the national reference frame and WGS 84.
- Determining by a computational datum transformation WGS 84 coordinates for all remaining points.

The two general groups of air navigation points which have to be surveyed are shown in Tab. 1-1.

AREA/EN-ROUTE COORDINATES	*AERODROME COORDINATES*
ATS/RNAV routes	ARP´s
Holding points	Thresholds
Radio NavAids	Radio NavAids - Non precision
Restricted/Prohibited/Danger areas	Radio NavAids - Precision
Obstacles - En route	Extended Runway centrelines
FIR boundaries	FAF´s
CTA, CTZ	Runway centreline
Other significant points	

Tab. 1-1. *Air navigation coordinates of interest*

1.5 Purpose of the manual

The purpose of this manual is to furnish guidance in the provision of geographical coordinates 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

Annex 14, Aerodromes, Volume II-Heliports

Annex 15, Aeronautical Information Services

The manual is intended to be amended from time to time. Users are invited to forward directly to EUROCONTROL suggestions for improvements or additions based on their practical experience when using this manual. Errors or discrepancies noticed in the manual should be brought to the attention of:

The WGS 84 Support Office

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CHAPTER 2

ACCURACY CONSIDERATIONS OF POINTS IN AVIATION

Traditional navigation techniques have relied upon the ability to fly to or from point navigation aids. Whilst 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 Range (VOR)/Distance Measuring Equipment (DME), Dual or multi DME and Global Navigation Satellite Systems (GNSS). Based on these data, RNAV systems generate a-ppropriate instructions to the autopilots which enable the aircraft to follow the planned route during departure, en-route and approach phases and, potentially with the introduction of GNSS, the landing phases.

For such operations the track actually flown by the aircraft depends upon the coordinates defining both the track and the location of ground navigation aids. With the advent of precision RNAV (RNP 1) routes and the extension of RNAV application to Terminal Area (TMA) procedures, greater precision is required and it is necessary to ensure that the data defining the track to be flown is of an accuracy and integrity which is consistent with the RNP requirements.

Definition of accuracy

The degree of conformity with a standard, or a value accepted as correct or true. 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.

Definition of integrity

The integrity of 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.

For the EUROCONTROL WGS 84 Implementation programme, the accuracy requirements are based upon a 95% confidence level. 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 $P = 1 - \exp(-c^2/2)$.

The Circular Error Probable (CEP) is the radius of the circle within which 50% of the measurements lie, that is, 1.1774σ. The radius within which 95% of the measurements lie is 2.448δ or 2.079 x CEP. The following Tab. 2-1 relates σ error-values, probable errors and probabilities in one, two and three dimensions.

Tab. 2-1. *Accuracy and probability*

Accuracy Expression	***One Dimensional Probability***	***Two Dim. Probability***	***Three Dim. 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)

The RNP types (see Tab. 2-2) 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 an input in defining route widths and traffic separation requirements, although RNP by itself is not sufficient basis for setting a separation standard.

Tab. 2-2. *RNP types*

	RNP 1	**RNP 4**	**RNP 12.6**	**RNP 20**
ACCURACY 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)

Definition of precision

A measure of the tendency of a set of random numbers to cluster about a number determined by the set. *Usual measure: Standard deviation with respect to the average or its reciprocal.*

Definition of resolution

The smallest difference between two adjacent values which can be represented in a measuring system. The number of decimal points or the scale of units to which a measured or calculated data item can be recorded, displayed or transferred.

The terms 'precision' and 'resolution' are often interchangeable in general use. Here it 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.

2.1 Type and classification

As already outlined in Chapter 1, Tab. 1-1 air navigation points can be divided into two groups:

- **Area** and/or **en-route** points
- **Aerodrome** points.

Besides this categorization air navigation points can be categorized by the type of positional data.

TYPES OF POSITIONAL DATA

Three types of positional data have been defined: *surveyed* points, *declared* points and *calculated* points.

Surveyed Points

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 standards developed for the implementation of WGS84. Communication facilities; gates; navaids; navigation check points; obstacles; obstructions and runway thresholds are usually surveyed points.

Declared Points

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. Airspace boundary points and oceanic entry and exit points are often declared points.

Calculated Points

A calculated point is a point in space which need not be specified explicitly in latitude and longitude, but which 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 waypoints, which are computed from the intersection of great circle routes, or cross radial fixes on great circle routes, are also calculated points, albeit that they are reported in latitude and longitude.

The data types are summarized in Tab. 2-3.

TYPE	*EXAMPLES*
Surveyed	Thresholds, navaids, obstructions, navigation check points, gate positions
Declared	FIR/UIR boundaries, reporting points, Prohibited/Restricted airspace
Calculated	Waypoints, FAF, FACF, MAPt, ARP

Tab. 2-3. *Types of positional data and examples*

Source of Survey / Positional Data

It is normally the responsibility of nominated technical branches within the Aviation Authority of a Contracting State to ensure the origination of the raw data required to be promulgated by the AIS (Aeronautical Information Service). On receipt of the raw data, the technical branches must check, record and edit the data so that it can be released in a standard format.

Raw AIS data containing positional information can originate from a number of different sources:

- **En Route**
  The location of navaids and communication facilities (-> surveyed) are normally provided by the owner/operator of the equipment.

- **Specific aerodrome information** (gate positions, obstructions, etc. -> surveyed) is normally provided by the owner/operator of the aerodrome.

- **Airspace divisions and restrictions** (-> declared, no survey) may be defined by the national aviation authority, national military authorities or other government bodies. Such divisions and restrictions may be either temporary or permanent.

- **SID, STAR, Approach / Holding procedures** -> calculated, no survey.
  These are usually determined by the air traffic service provider responsible for the procedure, in conjunction with the appropriate technical branch within the national aviation authority, who may have access to computer-aided modelling facilities to validate the procedure design.

CLASSIFICATION OF POSITIONAL DATA

The positional AIS data can be divided into a number of classes based upon the user requirements for accuracy and integrity.

More information on integrity in Chapter 5

Tab. 2-4 classifies data as function of accuracy, integrity and resolution.

Class	*Operation*	*Data Items (C=Calculated, S=Surveyed, D=Declared)*	*Accuracy (95%)*	*Integrity*	*Use (Resolution)*
1	Critical Path Points	Runway Threshold (S), FACF (C), Final Approach Passing Points (C), MAPt (S/C), FAF (S/C)	1 m	Critical CAT I: 3×10^{-8} CAT III: 8×10^{-10}	AIP: Threshold 1’’ 424: Waypoint,Runway Records 1’’ UDDF: Runway Centreline End, Displaced Threshold 0.0001’’
2	Navigation Check Points	Check Locations (S)	0.5 m	Essential 3×10^{-5}	AIP: INS Coordinates 0.01’’ 424: Gate Records 0.01’’
3	Landing and Take-Off	MLS (S) (Azimuth, Elevation, Back Azimuth, Datum), Helicopter Landing Areas (S)	1 m	Critical 3×10^{-8}	AIP: Helicopter Alighting Area, Radio Navigation /Landing Facilities 1’’ 424: MLS/Heliport Records 0.01’’ UDDF: Navaid 0.0001’’
4	Final Approach	DME/Precision (S)	3 m	Critical 3×10^{-8}	AIP: Radio Navigation /Landing Facilities 1’’ 424: VHF Navaid Records 0.01’’ UDDF: Navaid 0.0001’’
5	Obstructions	Obstructions in Approach/Take Off Areas (S)	3 m	Routine 1×10^{-3}	AIP: Obstruction 1°/1m from Threshold 424: Object in Approach/Primary Area 0.01’’
6	Departure, Arrival and Non-precision Approaches	DME (S), VOR (S), TACAN (S), Terminal NDB (S), Marker (S), SID/STAR Approach Waypoint (C)	30 m	Essential 1×10^{-5}	AIP: Radio Navigation /Landing Facilities 1’’ 424: VHF Navaid Records 0.01’’, Waypoint Records 1’’ UDDF: Navaid 0.0001’’
7	Reference Points and Obstructions	ARP (C), HRP (S), Obstructions at Aerodrome/Circling Area (S)	30 m	Routine 1×10^{-3}	AIP: ARP 1’’, Heliport 1’’, Obstruction 1°/1m from ARP 424: Airport Records 0.01’’ UDDF: ARP 0.0001’’, Object not in Approach/Primary Area 0.01’’
8	En-Route: Navaids, Holding Patterns, Routes and Designated Points	DME/N (S), VOR (S), TACAN (S), NDB (S), Routes and RNAV Waypoints (C), Aeronautical Ground Lights (S), Air Navigation Obstacles (S), Aircraft Stands (S)	100 m	Essential 1×10^{-5}	AIP: Navigation Facilities, En-Route 1’’, ATS/RNAV/Helicopter Routes 1’’, Ground Lights 1’’, Air Navigation Obstacles 1’’, Grid on Parking/Docking Chart 0.1’ 424: VHF Navaid/NDB Navaid /Waypoint/Gate Records 0.01’’ UDDF: Navaid 0.0001’’
9	Airspace Designations, Other AIP Coordinate Information	FIR/UIR, TMA, T/FIA, HPA, CTR, ATZ, TIZ, HPZ (D/C), Prohibited, Restricted and Danger Areas (D), Military Training Areas (D), Airport/Heliport/En-Route Communications (S), ILS, Decca, Loran, Consol Stations, Airways Marker (S)	100 m	Routine 1×10^{-3}	AIP: FIR/UIR/TMA/T/FIA/HPA /CTR/ATZ/TIZ/HPZ 1’’, Areas, ATS Communication Facilities, Special Navigation Systems SAR Units, Radar Stations 1’’ 424: Restrictive Airspace/FIR/UIR Records 0.01’’, Airport/Heliport /En-route Communication 0.01’’, Localizer/Glide Slope/Airways Marker Records =.01’’

Tab. 2-4. *Classification of positional data*
More detail on UDDF and ARINC 424 in Chapter 6

2.2 Requirements

For AIS data to be useable it must be accurate and, in this context, it can be sub-divided into two distinct categories:

- Evaluated data
- Reference data

Evaluated data include such information as positional data, elevation, runway length, declared distances, platform bearing characteristics and magnetic variation; while reference data include navaid identifiers, navaid frequencies, waypoint names, Rescue and Fire Fighting facilities, hours of operation and telephone numbers.

The accuracy requirement for the reference data is absolute - the information is either correct or it is not. Conversely, the degree of accuracy required of the evaluated data will vary depending upon the use to which the data are to be put. It follows that it is incumbent upon the users of the data to specify the accuracy requirements. This manual only addresses evaluated positional data but many of the procedures may be applied to other evaluated data and to reference data, if required.

The requirements on data, which shall be contained within the data processing procedures are explained in more detail in Chapter 5.

CHAPTER 3

THE GLOBAL WGS 84 COORDINATE SYSTEM

3.1 Definition of the WGS 84 coordinate system

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.

Origin and axes of the WGS 84 coordinate system are defined as following:

Origin = Earth's centre of mass

Z-Axis = 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.

X-Axis = 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.

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.

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 Fig. 3-1.

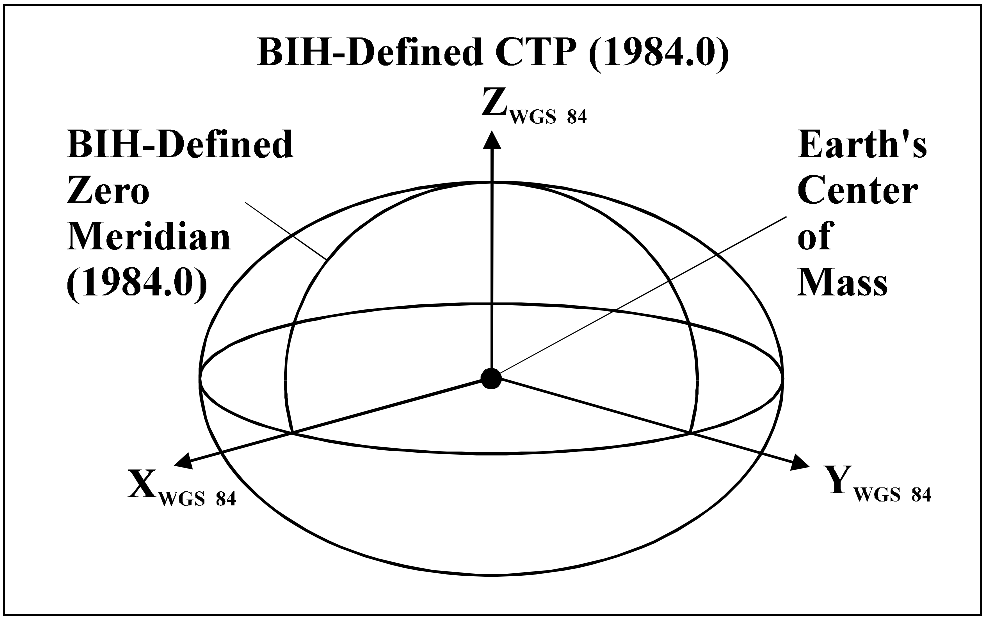


Fig. 3-1. *The WGS 84 coordinate system definition*

WGS 84 is an earth-fixed global reference frame, including an earth model. It is defined by a set of primary and secondary parameters.

The **primary** parameters are given in Tab. 3-1 and define the shape of an earth ellipsoid, its angular velocity, and the earth-mass which is included in the ellipsoid of reference.

Tab. 3-1. *Primary parameters of WGS 84*

PARAMETER	*NAME*	*WGS 84*
Semimajor axis	a	6378137 m
Flattening	f	1/298.257223563
Angular velocity	ω	7.292115×10^{-5} rad s^{-1}
Geocentric gravitational constant (Mass of earth's atmosphere included)	GM	398600.5 $km^3 s^{-2}$
Normalized 2nd degree zonal harmonic coefficient of the gravitational potential	$\bar{C}_{20}$	$-484.16685 \times 10^{-6}$

The **secondary** parameters define a detailed Earth Gravity Field Model (EGM) of the degree and order n=m=180.

The WGS 84 EGM through n=m=180 is to be used when calculating WGS 84 geoid heights, WGS 84 gravity disturbance components, and WGS 84 1° x 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 EGM 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

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 Fig. 3-2).

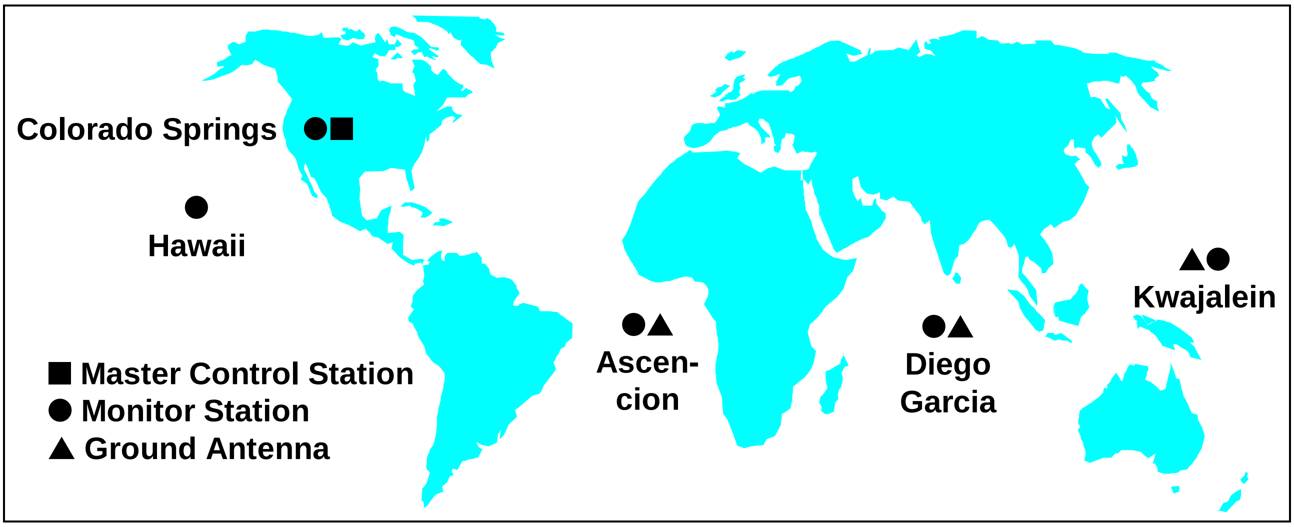


Fig. 3-2. *Realization of origin and orientation of WGS 84*

Historically the coordinates of the GPS tracking sites have been determined by the use of doppler measurements to the TRANSIT satellite navigation system. Long observation periods of data 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 once again that errors can propagate in the procedures used to realize reference frames.

Accuracy of WGS 84 coordinates

The accuracy (one sigma) 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 are:

Horizontal	$\sigma_\phi = \sigma_\lambda = \pm 1\text{m}$	(1σ)
Vertical	$\sigma_h = \pm 1 \ldots 2\text{ m}$	(1σ)

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

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, the lack (in general) 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.

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 CAT III are to be used in future. Precision CAT III needs a vertical accuracy of 0.6 m (horizontal: 6.0 m), which cannot be supplied by WGS 84 according to its accuracy definition, but for instance by ITRF.

For more on ITRF see Appendix C

CHAPTER 4

A GUIDE TO GET WGS 84 COORDINATES

The following chapter is intended to be a guide with ”recipes” for users to produce WGS 84 coordinates. It is written in such a way that the user of the manual is guided step by step to decide, dependent on the quality and/or existence of his data, what he shall do to get WGS 84 coordinates.

So the first decision, which has to be made is to answer the following question:

Are sufficiently accurate coordinates available?

Yes If these coordinates are available in a local reference frame, then proceed to *Case 1 (= Chapter 4.1)* of this chapter. If the available coordinates have been digitized from maps, then proceed to *Case 3 (= Chapter 4.3)* of this chapter.

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 minimum requirements for coordinates the surveyor must ensure that:

See Chapter 2, Tab. 2-4 and Chapter 5 if the coordinates fulfil the accuracy and integrity requirements. See also Ref. [6] and [15] for further information. See Ref. [3] and [5] for help on performing a local survey and redundant measurements

- point labels have not been interchanged or misidentified,
- that the coordinates can be verified by aid of redundant measurements,
- that the accuracy is predictable and sufficient.

No If no accurate coordinates are available or if, for example, the integrity requirements can not be fulfilled a re-survey with related field work must be performed. The different methods of performing this re-survey to provide accurate WGS 84 coordinates are explained under *Case 2 (= Chapter 4.2).*

4.1 Case 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 the knowledge of the transformation parameters, the type of coordinate system, and can be used as stand-alone or combined methods.

4.1.1 Checking the type of coordinate system

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 formulas of Chapter 4.1.2 to determine the WGS 84 coordinates. Several software programs exist to support this procedure, e.g. the DATUM program.

DATUM *(Ref. [2]) performs coordinate transformations between a variety of existing geodetic reference frames and WGS 84*

No Use the GPS surveying technique to survey 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 get additional WGS 84 coordinates for determining all seven Helmert transformation parameters (using the Inverse Helmert transformation). However, in practice it is usual to use as many common points as possible to obtain the best estimate of the parameters by least squares (e.g., Ref. [1]).

See Chapter 4.2.2, Appendix A and Tab. A-2 for information on GPS surveying

See Appendix E for a detailed description of the Helmert Formula

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

$$\underbrace{\begin{bmatrix} \Delta X \\ \Delta Y \\ \Delta Z \end{bmatrix}}_{\text{Shift of Origin}} = \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{WGS84} - \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{Local}$$

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

See Appendix E for a detailed description of the Helmert Formula

After determining all necessary transformation parameters, proceed as explained in the beginning of chapter 4.1.1.

The way of referencing a local (e.g. relative) and sufficiently accurate GPS aerodrome survey to WGS 84 by simply measuring the coordinate differences between one aerodrome reference point to a known and monumented WGS 84 station is called *direct geodetic connection*. On applying this procedure all the airport coordinates can be directly transformed to WGS 84. However the problem is that even in Europe not many geodetic stations exist for which accurate WGS 84 coordinates are known. Therefore it is recommended in Europe to use, if available, ETRF 89 stations for this purpose. In most European countries many ETRF 89 stations exist. If no ETRF station is near the 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.

See Chapter 2, Tab. 2-4 if the coordinates fulfil the accuracy require-ments.
See Chapter 4.2.2, Appendix A and Tab. A-2 for information on GPS surveying.

For more information on ETRF see Appendix D

4.1.2 Datum transformations

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

- *Helmert´s 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.
  Helmert's formula can also be applied for spatial ellipsoidal coordinates ϕ, λ, h by transforming from ellipsoidal coordinates to rectangular coordinates and vice versa.

See Appendix E for a detailed description of these datum trans-formations formulas
See Tab. B-1 for a list of reference ellipsoids and parameters

- *Standard Molodensky Formula* to solve the transformation in curvilinear coordinates ϕ, λ, h.

See Tab. E-1 for a list of WGS 84 trans-formation parameters

- *Multiple Regression Equation* approach to account for the non-linear distortion in the local geodetic datum.

4.1.3 Accuracy considerations

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. It has to be checked by the user, in particular, whether the resulting coordinate accuracy still meets the requirements. Furthermore, this quality control may be difficult to perform.

There are several reasons known for this loss of accuracy:

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 NavAid's original coordinates may not be sufficient, and in many cases the accuracy of the datum parameters is undefined.

Another reason for degrading the accuracy of transformed coordinates is that there may be a slight distortion in the national network in the area under consideration.

Limitations of transformations

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 country are, in practice, not constant but vary with location in the geodetic network.

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. E.g. the orientation angles of a datum could be published typically as, say,: 0.5 " $\pm$ 0.3 ".

The predicted error, or uncertainty, is often larger than the value itself.

Fig. 4-1 shows in a qualitative manner how the errors in transformation procedures propagate into transformed coordinates.

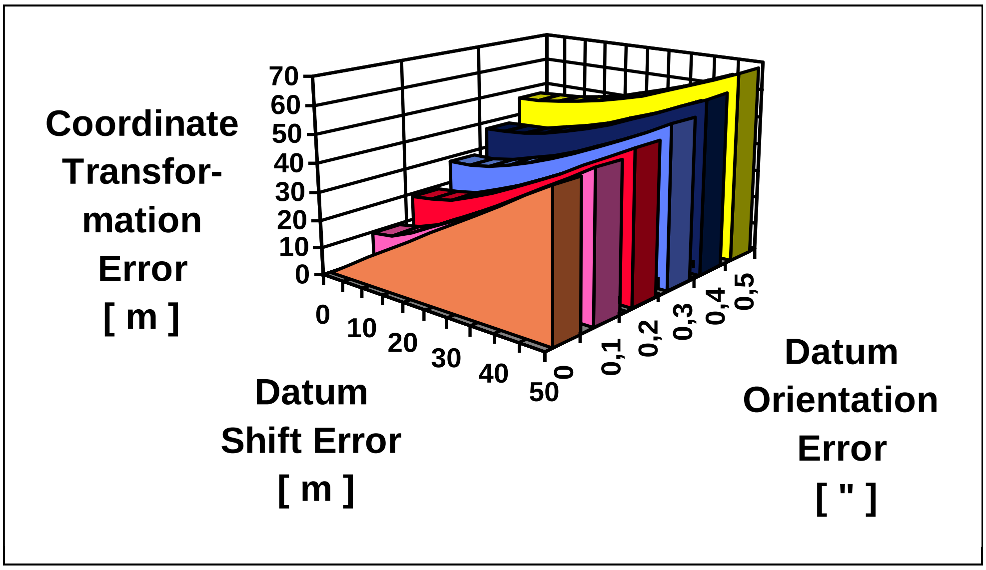


Fig. 4-1. *Error propagation in datum transformation*

The error sources in a datum transformation are errors in the shift parameters, in the orientation parameters and in 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 meter level accuracy in WGS 84. Here the difference between absolute and relative point accuracy has to be considered.

4.2 Case 2: No (sufficiently accurate) coordinates are available

If there are no (adequately) precise coordinates available, a new field survey must be performed using one of the following techniques:

- Conventional surveying
- GPS surveying
- Photogrammetry

Help in deciding, which of the above techniques is the most efficient one for the new field survey may be gained from the following:-

- Use the GPS technique for surveying limited and relatively small areas in a very economical way.
- Use the photogrammetric technique, if the area to be surveyed is very large.
- Use conventional surveying, if the area to be surveyed contains a lot of obstructions, which would lead to GPS signal losses or multipath.

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.

4.2.1 Determination by conventional terrestrial surveying

Fig. 4-2 shows, how WGS 84 coordinates can be obtained by terrestrial surveying.

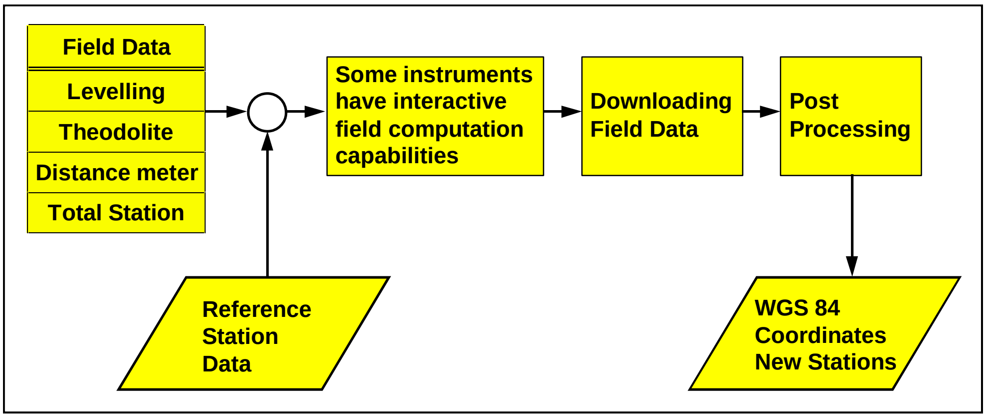


Fig. 4-2. *From terrestrial surveying data to WGS 84 coordinates*

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 postprocessing is carried out.

For more information on conventional surveying see Appendix F.

Before the derived coordinates may enter the survey data base they have to be quality controlled and integrity checks have to be performed. Various graphic visualizations of data and results can be done.

See Chapter 5.2 for more on quality control. See Chapter 5.3 for more on integrity check

4.2.2 Determination by GPS surveying

As already outlined in the beginning of this section 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. The method has the advantages of 24-hour all-weather operations, ease of use, speed, very economical, high accuracy and, most importantly, direct compatibility with the WGS 84 datum.

For the choice of the GPS surveying technique, i.e. static, rapid static, kinematic survey, etc., which depends to a great extent on the desired accuracy see Appendix A and Tab. A-2

GPS receivers store the field data. After finishing the survey the data have to be downloaded to a computer where they are postprocessed 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 may enter the survey data base they have to be quality controlled and integrity checks have to be performed. Various graphic visualizations of data and results can be performed.

See Chapter 5.2 for more on quality control. See Chapter 5.3 for more on integrity check

4.2.3 Determination by aerophotogrammetry

Fig. 4-3 shows the determination of WGS 84 coordinates by using photogrammetric flights.

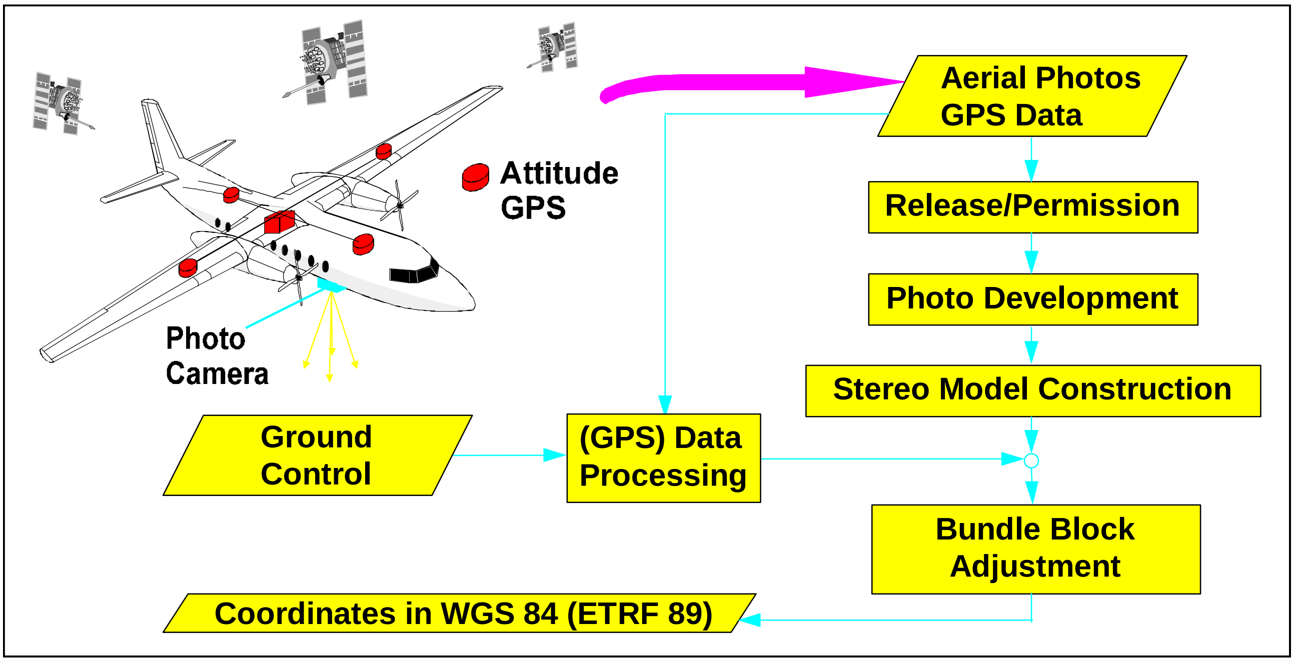


Fig. 4-3. *From results of photo-flights to WGS 84 coordinates*

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. So 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.

For more information on aerophotogrammetry and the minimization of ground control stations see Appendix F

After obtaining permission to release photo data (if necessary), they are 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.

Again, before the derived coordinates may enter the survey data base they have to be quality controlled and integrity checks have to be performed. Various graphic visualization of data and results can then be done.

See Chapter 5.2 for more on quality control. See Chapter 5.3 for more on integrity check

4.3 Case 3: Digitized coordinates from maps are available

This section helps the user to transform coordinates to WGS 84, if the coordinates are available from digitized maps. After some remarks on the restrictions of digitized maps the user is guided through the transformation process in Chapter 4.3.2.

4.3.1 Restrictions

While digitized data has no inherent scale information, the accuracy of the data is clearly limited by the corresponding accuracy of the analogue map from which it was originally extracted, and of the digitising process involved. A new analogue map may be printed at a larger scale than that of the original map, but in doing so one does not increase the accuracy to that normally associated with the larger scale. The problem is further compounded by the frequent revision and updating of the data base 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.

The most important drawback of digitized maps, however, is the very nature of an analogue data base. 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 to these, one must consider the more significant projection scale and orientation errors inherent to 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 be substantial.

So when digitizing coordinates from maps the following points should be considered:

- Check how the map was established, by which technique (from analogue data/digitizing of other maps, from digital data, etc.)
- In order to convert the Northings and Eastings to geographical coordinates, it is necessary to know the exact formulas of the map projection.
- Furthermore, it is necessary to know the original datum of the projected coordinates as well as the new one, when transforming.
- Datum coordinate transformation can only be applied after converting map projection coordinates to geodetic coordinates.

- The resulting accuracy of such coordinates should be checked and verified in order to decide whether the anticipated accuracy needs have been met.

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

- different zero points (tide gauges),

- different type of height systems. (There are not only orthometric heights. For example, you find, so-called normal heights in Eastern Europe.)

4.3.2 Transformations

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

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

No If this question cannot be answered, then one has to proceed to *Chapter 4.2* and to perform a re-survey.

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.

See Appendix G for different types of map projections

Note: 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 and N is the geoid height.

In general, only the orthometric height is known (and found also on maps). The geoid height has to be taken from a digital model (if available).

However, according to Ref. [2], 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 formulas. By assigning heights ranging from zero metres to 8000 m, it was concluded that the effect on both latitude and longitude was negligible (less than 15cm at 8000 m). Consequently, for a point of known latitude and longitude, but unknown (orthometric) height, an arbitrary height of zero metres could be assigned without significantly affecting the transformation.

Because national surveying agencies are using different kind of reference ellipsoids, the next step is to determine this reference ellipsoid in order to be able 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.

The transformation from the local datum to WGS 84 can be done following the explanation in Chapter 4.1.

CHAPTER 5

QUALITY ASSURANCE AND INTEGRITY

5.1 Quality Definitions

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 there personal experience as a consumer. The following definitions and descriptions are included to establish consistency.

Procedures

These describe the method used:-

- how the responsibilities for the task should be assigned.
- what should be achieved in the tasks.
- what should be recorded as the associated quality record.

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 do' 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: An ***entity*** *is an item which can be individually described and considered* ***(ISO 8402)****.*

Quality can be described as the ability of a product to consistently meet its stated requirements, that the product is fit 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' defined below.

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).***

Once a method has been proven to produce the required product successfully, a system is required that can assure the method or methods are followed correctly each time the process is repeated. This is quality assurance and is achieved through the use of a Quality System.

All activities and functions which affect the level of quality of a product are of concern to Quality Assurance.

Quality Level

The extent to which the customers needs have been met. A quality level of 100% means that there has been a complete conformance to specification <u>every</u> time.

Quality Management

The implementation of Quality Assurance (QA).

Quality Record

Documented evidence of tasks carried out which demonstrate that the required results have been achieved and providing sufficent links to other quality records to ensure traceability.

Quality System

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

Standard

The minimum specifications that must be met to fulfill the stated requirements.

A quality system provides the managment control to assure the required quality thresholds are achieved. These quality thresholds or specifications must be pre defined. Hence the need for a ‘standard document’.

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)***

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

Verification

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

Work instructions

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

Some steps may seem too obvious to be documented. But these can be items frequently overlooked due to familiarity to some but not to other personnel.

5.2 QA implementation

Why do we need a Quality Assurance system?

The objective of the WGS 84 Implementation Programme is to produce coordinate data referenced to a common datum in which a high degree of confidence can be placed on 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 is aqcuired. This can be achieved by quality assurance.

QA and WGS 84 Implementation

QA is about preventing errors occurring rather than fixing them. A method has to be designed that will give the required end product when followed correctly. To do this it is essential to understand the requirement, if not we cannot guarantee the product. A Standard needs to be developed, for example Ref. [6]. The same method must be used each time, i.e. we must be able to control each step of the operation. This is fine when looking at the prospect of one survey company working on one type of survey. The extent of the implementation of WGS 84 will vary for different Administrations and similarly for the Agency monitoring the overall progress of the programme. The various tasks involved need to be identified and managed efficiently. This is done by using a Quality System (QS) or Quality Management System (QMS). An example of the basic elements of such a system follows:-

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

- **Planning/Procedures.** Identifying the tasks to be done. Developing procedures necessary for the production process.

- **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 cause of a problem to be located.

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

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

Fig. 5-1 shows a basic structure of a quality system.

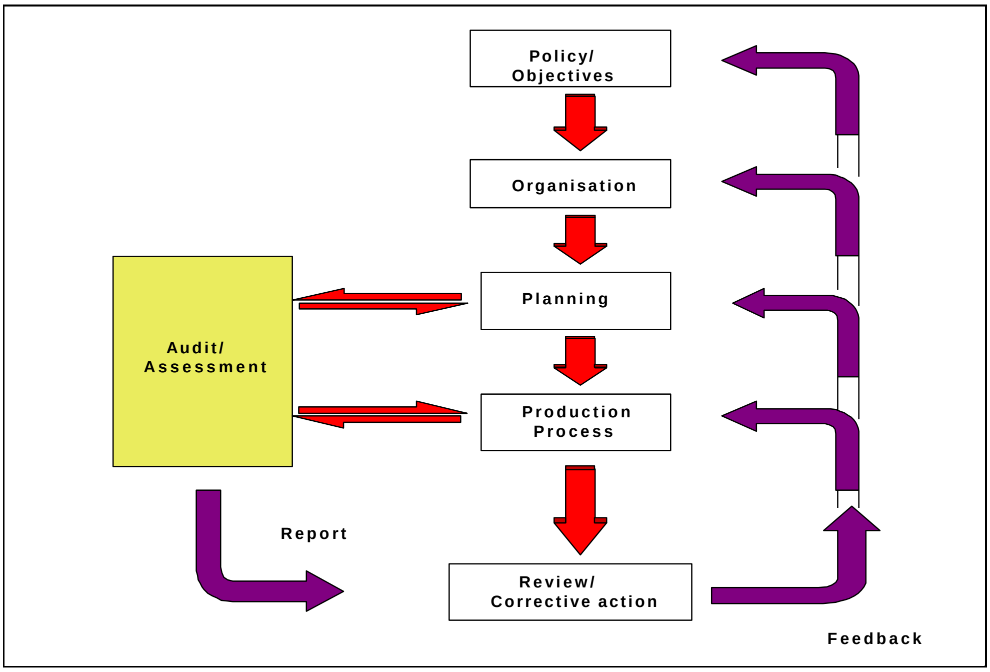


Fig. 5-1. *QA loop*

This is expanded further in the following Fig. 5-2.

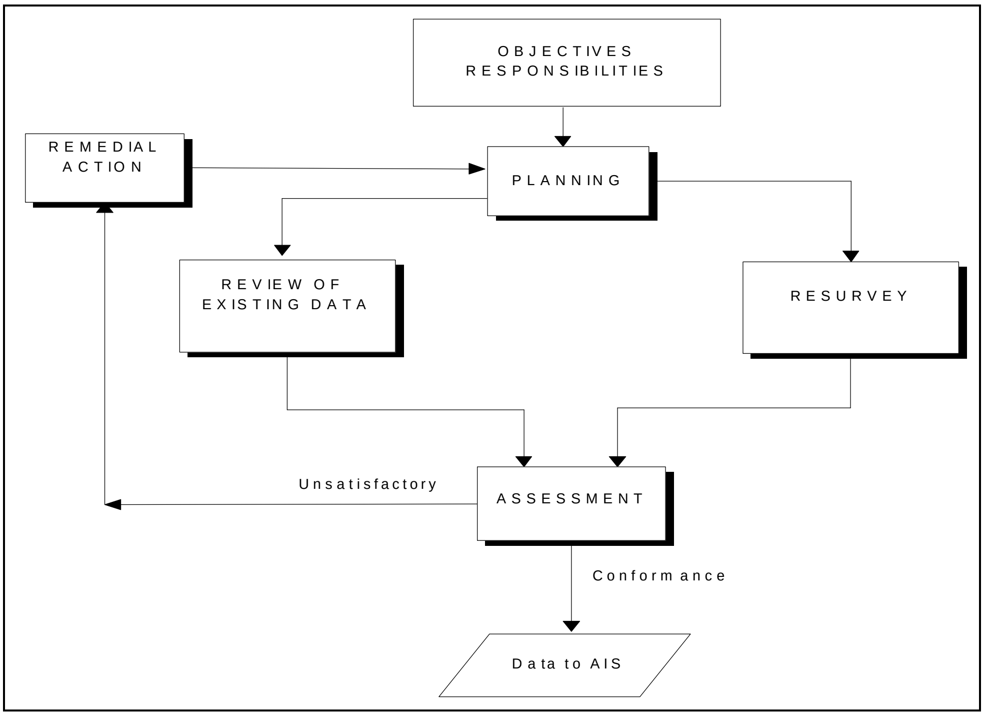


Fig. 5-2. *Model of State Quality Plan*

Fig. 5-2 is an example QMS structure applicable to the QA of the acquisiton of new data described as follows:-

1. The objective and the responsibilities as set up within the National Administration.

2. The various planning tasks that are needed, including:-

    - the decision as to whether to use existing data or to resurvey.
    - the type of survey, geodetic, aerodrome or en route.
    - the accuracy requirements.
    - the briefing of survey contractors on these requirements, safety issues and evaluating their suitability before awarding contracts.

3. Having received a completed survey, the assessment process (or audit).There are two outcomes from the assessment;

    I. conformance in which the data can then proceed through to transport onwards.

    II. non conformance where the process flow then should loop back to the planning via a corrective (or remedial) action procedure.

Safety

A Quality System must take account of safety issues. Involvement in safety has various legal positions to be considered, and will vary within each Administration.

However any Quality System 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 are to do, and on any local procedures that they should be aware of. This is particularly important where work about airfields is concerned.

Prior to awarding any contracts evaluation of the survey company's experience should be considered. Such as, have they conducted any airfield programmes before. To follow the requirements of a Quality Management System quality records should be kept, such as a checklist of the briefing of the survey team. For example, if there is no acknowledgement that the surveyors have been briefed on the airport safety requirements then they should not proceed.

Quality Plans

Note that the above QMS has been designed purely for the QA of the origin of WGS 84 data, largely relying on the acquisition by survey crews. The scope of this system does not extend to include the management of all the navigation data processes that may be the responsibility of an AIS department. However it is possible for this QMS to function as a subsidiary element within a total QS having a wider scope. The demoted QMS is refered to as a Quality Plan. For example the QMS described in Fig. 5-2 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 QP's such as for managing the onward flow of data via data base storage and publication. Fig. 5-3 gives an example relationship of such a total data management system.

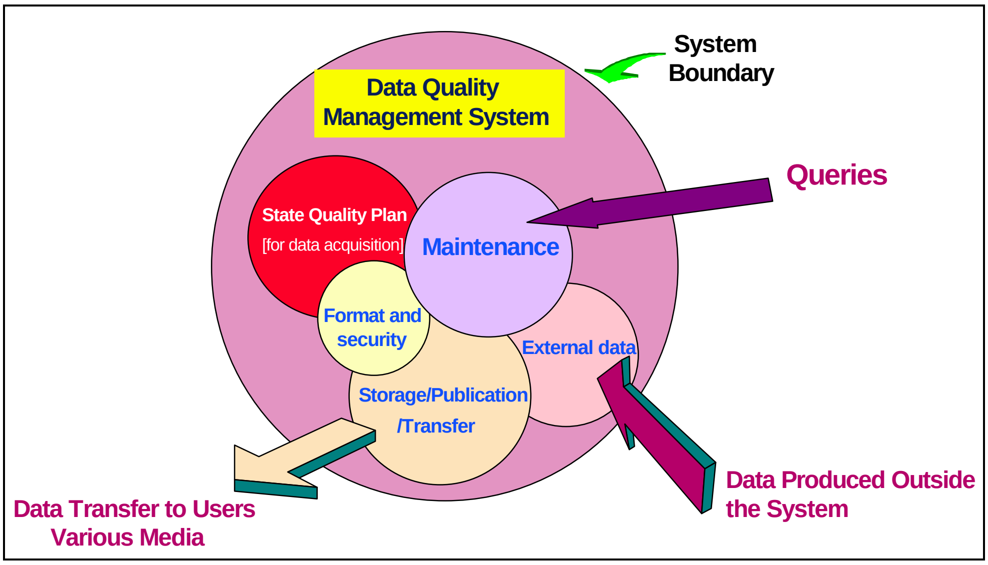


Fig. 5-3. *Example of Quality Plans within a Quality System*

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. Whilst the CRC will be used by receivers of data to confirm correct receipt, it is not sufficient to define quality of data. A record must, therefore, be kept of all changes. A sample set of data required could be elements of the following:

- Data accuracy
- Origin
- Details of changes made to the data;
- The reason for the change;
- References associated with the change;
- The source of the change;
- The identity of the person making the change
- Date of the change.

Procedures to ensure traceability

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

Such records may be electronic or paper-based, although certain change information must remain with the data item throughout the navigation data cycle, to provide the unbroken trail and must, wherever possible, be stored in an associated field or record.

5.3 Integrity

Definition of integrity

The integrity of 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 application.

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 3×10^{-8} means that an undetected corruption can be expected in no more than three data items 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 is accurate without a further verification of the data from the point at which integrity can be confirmed.

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 guidance in cruise.

It is important to note that a lower accuracy does not necessarily imply a lower integrity requirement.

Requirement for integrity

The use to which a data item is put also forms the basis for determining its integrity requirement. A data classification is proposed which defines requirements based upon the potential risk resulting from corruption of the data:

A classification of positional data with respect to accuracy and integrity can be found in Tab. 2-4

Critical Data: There is a **high probability** that, as a result of using corrupted Critical Data, an aircraft would be placed in a life threatening position.

Essential Data: There is a **low probability** that, as a result of using corrupted Essential Data, an aircraft would be placed in a life threatening position.

Routine Data: There is a very low probability that, as a result of using corrupted Routine Data, an aircraft would be placed in a life threatening position.

To each of these types of data an integrity requirement has been assigned as follows:

Critical: This level is given to the runway data which defines the landing point. For these data, two levels of criticality have been defined . These have been related to the approach and landing criteria categories. The levels of integrity have been derived from the integrity requirements for autoland and are defined to ensure that the overall process, of which data is only a part, has the required integrity. Thus:

CAT I : 3×10^{-8} CAT III : 8×10^{-10}

Essential: This level is assigned to points which, whilst an error can in itself result in an aircraft being outside of the envelope required, this excursion does not necessarily result in a catastrophic failure. Examples include en route navigation aids. The integrity requirement is defined as: 1×10^{-5}

Routine : This level is assigned to data for which errors do not affect the navigation performance. These include FIR boundaries and obstructions. The reason why obstruction data can be held with a relatively low level of integrity is that whilst the latter data needs to be accurate at the time the procedures are derived, any subsequent corruption should have no impact on the safety of the aircraft on condition that it conforms to the procedure requirements. The integrity requirement for routine data is: 1×10^{-3}

Procedures to ensure integrity

The accuracy of data is determined at the point where the data originates. In the case of surveyed data, the Procedures necessary to ensure accuracy are being addressed in the WGS 84 Implementation Programme. Declared points must be declared to the accuracy required by the data model. Procedures for calculating points must not only be detailed in the Quality Framework and take account of the accuracy of the source data but must also ensure that subsequent mathematical manipulation maintains the accuracy set by the data model requirements.

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.

Manual Data Entry

The transfer of data from written or printed form into the format the data is stored in a computer, is the *greatest potential source of error in the entire process*. If end to end integrity, at the required levels, is to be achieved careful consideration must be given to the means by which this transfer is to be performed and verified.

Validation Checks

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

Software Aspects

Whenever data are manipulated by a computer program, even if it is simply to extract an item from a data base 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.

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

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.

Data Transfer Aspects

The risk to data, while being written to or read from magnetic/optical storage media, depends upon 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 operation.

It is normal practice to provide this protection and assurance by the use of CRC, see Chapter 6.2

Data Use Aspects

Consideration must be given as to whether the Procedures for the production and delivery of data to the airborne Flight Management System (FMS) have provided the necessary level of integrity when the data are eventually extracted from the FMS data base for final use. The probability of corruption after delivery into the FMS will depend upon the design of the FMS itself.

System Environment Aspects

An important reason for not achieving the required timescales for data change notification could be a failure of a computer system used within the process of dissemination. The effects of such system failure can only be reduced by the use of independent, geographically separated back-up systems and regular archiving.

CHAPTER 6

DELIVERABLES

6.1 Survey Reporting Requirements

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 the programme 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.

It is therefore essential that the surveyors record all the necessary information that will be required. To ensure this a minimum survey report specification should be given in the Surveying Standard. This way all survey work undertaken to determine the coordinates of navigation facilities can be reported in a predefined format. Where existing national reporting practice differs from that shown in the Surveying Standard the National Administration may make a case in support of the national standard where this can be shown to be compatible.

Reference to the Standard should provide the information on those items which will be considered mandatory in an assessment, and those which should be available (recommended) but not mandatory.

An advantage in having consistency of report formats will be seen in the assessment phase. Checklists drawn up based on the Standard format and cross referenced to the mandatory points would facilitate an efficient audit process particularly where large numbers of surveys are involved. A further advantage can also be seen in providing the surveyors with the checklists prior to operations as a guidance document for the reporting requirement.

6.2 A basic reporting structure

The following description is derived from the reporting format specified in Ref. [6].

6.2.1 Types of survey

Three types of reporting formats.

- Geodetic
- En Route
- Aerodrome

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

6.2.2 Common reporting elements

A number of topics are common to each of the report formats.

- **Historical data** should describe the general survey information
  - its purpose,
  - the date,
  - the surveyors names and the company.
- **Survey method used** This is the actual way the survey was carried out not just a description of the theory behind the technique used.
- **Diagrams** Where relevant, diagrams should be included such as for station descriptions, control networks, threshold descriptions.
- **QC report**. The Quality Control (QC) report should provide information of 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.
- **Observations** Records of the actual observations should be provided in a separate volume. Cross references should be made to the survey report.

6.2.3 Aerodrome survey report format example.

The next list shows a complete reporting format specified in Ref. [6] for an aerodrome 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 which gives equipment calibration detail, and describes the methods used to check the survey. In addition evidence should be provided to show that the accuracy requirements have been met.

8. The actual observations should be provided in a separate volume indexed so that cross references can be included in the report.

The above list provides the sections headings from which a set of checklists can be developed. What is useful about the prepared checklists is not only their application in the audit or assessment process, but that they can be used in briefing the survey teams on the requirements of the Surveying Standard. The benefits being consistency for the Administration.

6.3 Formats, Standard Algorithms and Working Practices

This section deals with suggested data formats, standard algorithms and working practices.

6.3.1 Formats

- Universal Data Delivery Format (UDDF) - Surveying data
- Annex 15 ICAO (AIS data in printed form)
- ARINC 424 (Navigation reference data)

6.3.2 Algorithms

- CRC Algorithm
- DO 201: Waypoint coordinates
- ARINC 424: Path Terminators

6.3.3 UDDF

The UDDF details a digital format that can be used when delivering surveyed data. It accommodates fields for various aeronautical data including aerodrome/heliport runway, navigation aid and obstacle, in a standard ASCII format.

Annex 15 to the Convention on International Civil Aviation details the format to be used when reporting AIS data in printed form. Standardization of a transfer format for an Electronic AIP (E-AIP) is currently under consideration.

ARINC 424 details the format to be used when preparing navigation reference data tapes/disks for merging with operational FMS software, flight planning system software and simulator software and can be considered to be the standard format for electronic transfer of navigation data.

A CRC algorithm ensures integrity to a certain level. DO 201 provides recommended priorities for defining, and rules for the calculation of waypoint coordinates.

Fig. 6-1 shows a **navigational aid data** example for the UDDF format.

```
0        1         2         3         4         5         6         7
12345678901234567890123456789012345678901234567890123456789012345678901..
.

|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|       |     |      |
```

Fig. 6-1. *UDDF format for navigational aid data*

The 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 - NONPRECISION AREA NAVIGATION APPROACH SURVEY
   PAA - PRECISION AREA NAVIGATION APPROACH SURVEY
   SPL - SPECIAL SURVEY

   (SPECIFIC SURVEY REQUIREMENTS FOR EACH SURVEY TYPE CAN BE FOUND IN "FAA No. 405 - STANDARDS FOR AERONAUTICAL SURVEYS AND RELATED PRODUCTS.")

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

6.3.4 Data Management

The acquisition of WGS 84 navigation data has involved resurveying of ground points and/or the conversion of existing data by mathematical transformation. It is essential that once this work is completed the data and associated computed/derived points be held in some form of data management system. Any entry process to such a system must be able to confirm to users that the data is correct when the entry process is completed and that it remains so. This means that two issues must be addressed before any integrity measures can be applied:-

- the data is accurate.

- the data is referenced to the correct location i.e. associated identifier is present.

When entering data into the data base it will be necessary

- to confirm them against the original survey data
- to enter the data under strict quality control procedures
- to verify entered data.

6.3.5 Validation and Verification of WGS 84 Data

Of primary importance to the WGS 84 project is the point of origin of survey data. It has been recommended that source data be stored in some form of database. This can be a master database holding all the navigation coordinate data or even a source database with records that contain additional supporting information such as more detailed survey and geodetic information. How this data should be entered into such a system before integrity measures have been applied is considered here.

6.3.5.1 Methods

Manual Entry

The transfer of data from written or printed form onto electronic media. The procedures involved can vary but include one or more of the following:-

- multiple entry by different operators
- repeated entries by the same operator.
- use of automatic (software) comparison methods to check entries against each other.

Careful consideration must be given to the means by which the transfer from printed to electronic form is to be performed and verified, if end to end integrity is to be achieved.

Software

Software methods include the following:-

1. Comparison checks, for example:

- 100 % comparison - between multiple entries.
- data is within specified range limits.
- location is within a valid boundary.

2. Computation of cyclic redundancy check values.

- data has not been corrupted since the CRC was applied.

Whenever electronic data is being manipulated there is the risk of exposure to computer viruses either directly corrupting data or affecting the associated software.

Independent derivation.

Validation of data by using a separate derivation method, such as:-

- duplicating survey work. e.g. as a quality check on existing data.
- comparison with existing data by mathematical transformation.

Graphic overlays.

Production of overlay plots that can be compared with master charts.

Test procedures.

Apply data to check or test procedure and evaluate the results.

6.3.5.2 Validation

The activity whereby a data item is checked as having a value which is fully applicable to the identity ascribed to the data item.

Validation checks provide some assurance that data have been correctly entered, maintained or transferred and they can assist the checking of the integrity of the data to a limited degree. However, validation checks cannot be used to improve the reported data quality. Their primary use is to filter out gross errors. These checks include:-

- **range limit -** ensure that data fields which have been accorded specific ranges of values, do not hold data values outside that range.

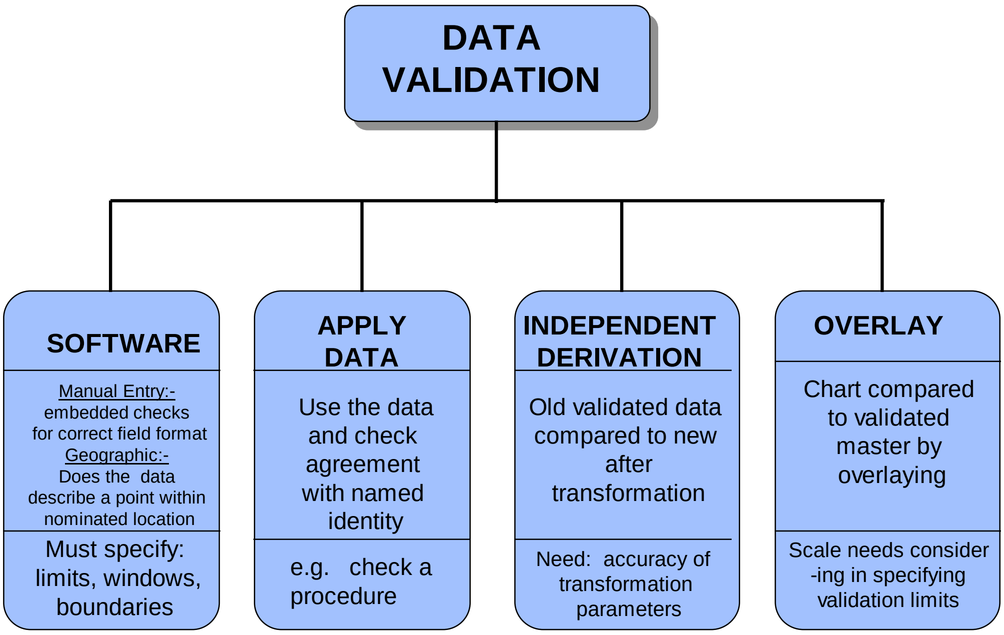
- **related record/field -** ensure that appropriate data are held in related fields or records. e.g. every survey point needs to be supported by an identifier, description, latitude, longitude, and height data.

- **data item relationship -** such as collinearity, elevation and geographical vicinity checks.

Validation checks are often confused with verification, they are two different processes. Both are important as they can take place at different points of the data management operation. Normally, it is better to validate before verifying, as the latter can be a more time consuming process. It would therefore be more efficient to eliminate any gross errors prior to verification.

Survey quality checks can be made by comparison with independent derivation, e.g. existing data may be used where parameters have been derived to transform existing data from the local datum to the WGS 84 reference system.

Manual data entry methods can be made using software to check operator entries for correct field format, range limitations, expected geographical location.



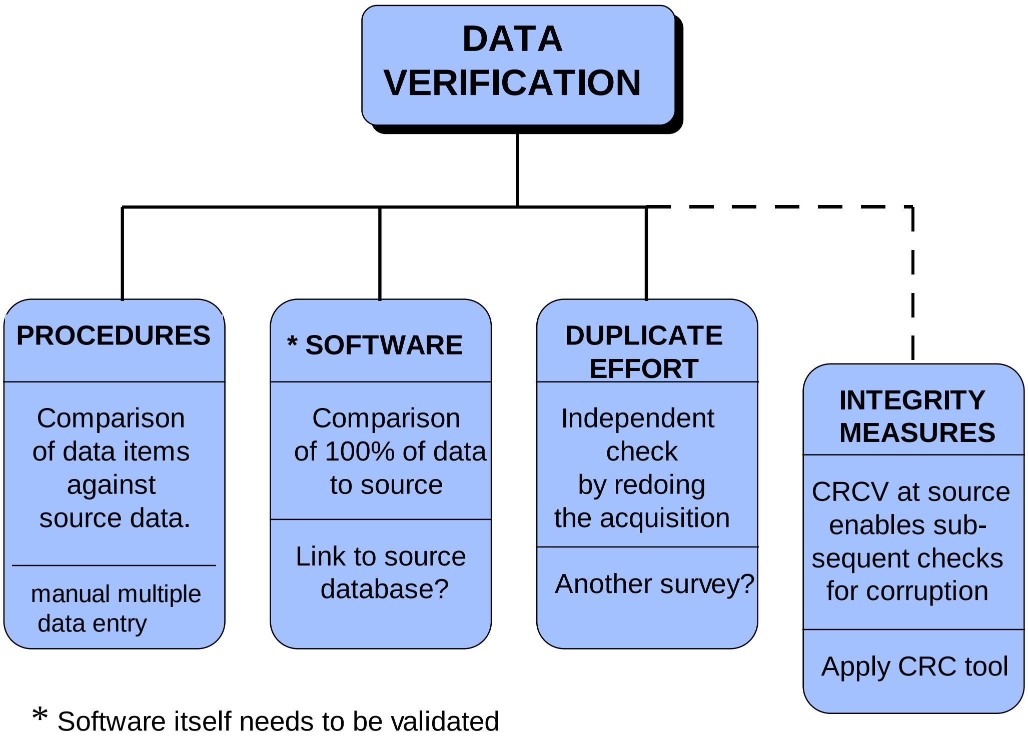
6.3.5.3 Verification

The activity whereby the value accorded to a data item is checked against the source of that value.

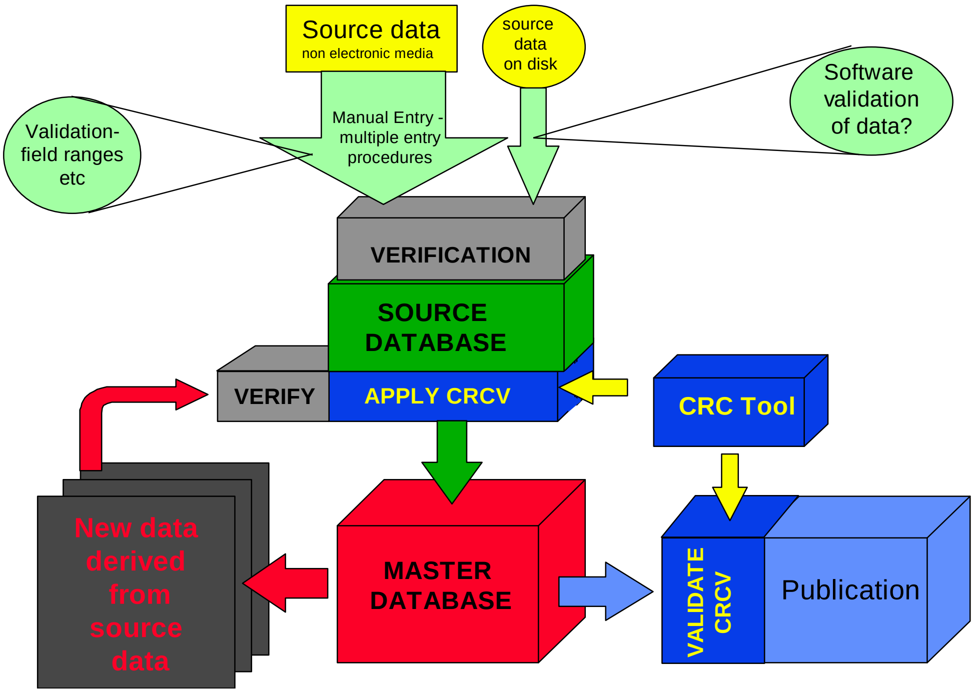
Here we have to achieve 100% comparison to the source data. Where manual data entry is used, software should be used to check consistency of multiple data entry methods.

Verification is a process for checking the integrity of a data item. It can take place when data are input into a database, e.g. a visual check of the input data against the original source document by an independent checker, or an automatic check of the same data which is entered two or more times by one or more data entry operators (double, triple entry methods). Recomputation and confirmation of CRC values is also a form of verification check.

Note that the use of a CRC tool can only be effective in this instance if the CRCV's are applied at the point of origin by the data supplier i.e. verification has taken place at this point. Validating a CRCV only confirms that the data has not changed since the CRCV was derived and appended.



EXAMPLE WGS 84 DATA MANAGEMENT OVERVIEW

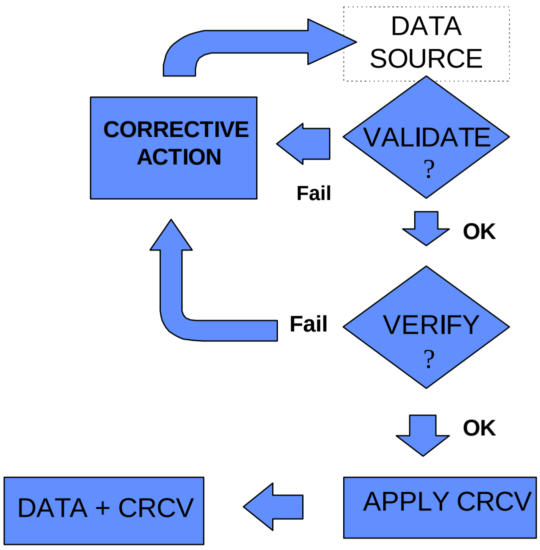


6.3.5.4 Corrective Action

Any procedures established to validate and verify must also include a procedure to ensure that where errors are identified these are dealt with by corrective action i.e.

- ensure that the corrupt data is not stored or passed on.
- investigate the identified problem.

For example:-



6.4 Cyclic Redundancy Check (CRC)

The 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 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 corrupted and the data is 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. Typically, the highest level of integrity will be achieved for the latitude, longitude and identifier with a 32 bit CRC.

In summary, the characteristics of a CRC are:

- Better than check sums or parity bits for error detection
- Elements of generating a CRC:

  - Data block divided by a generating polynomial

  - CRC = Resulting Remainder

  - CRC tagged onto the end of the data block

  - No errors: Remainder from division should be zero

- Assurance of error detection (see relationship integrity <> CRC length, Tab. 6-1)

6.4.1 Standard CRCAlgorithms

16 bit

CRC-16 (ANSI) $1 + x^2 + x^{15} + x^{16}$

CRC-CCITT $1 + x^5 + x^{12} + x^{16}$

32 bit

CRC-(LAN)

$$1 + x^2 + x^4 + x^5 + x^7 + x^8 + x^{10} + x^{11} + x^{16} + x^{23} + x^{26} + x^{32}$$

CRC-MLS

$$1 + x^1 + x^3 + x^4 + x^8 + x^9 + x^{13} + x^{14} + x^{31} + x^{32}$$

CRC - 32Q

$$1 + x + x^3 + x^5 + x^7 + x^8 + x^{14} + x^{16} + x^{22} + x^{24} + x^{31} + x^{32}$$

The algorithm providing the CRC must be quality controlled and meet defined standards requirements. The rules for the application of the CRC are still being developed. However, in order to ensure the integrity is assured from the start of the surveys it is proposed to provide a CRC algorithm for use by States. If this subsequently needs to be modified, a conversion programme will be provided.

Note that the CRC - CCITT and CRC -32Q algorithms have been suggested for uniform application by States worldwide. (Ref 7.3.2.5 ICAO WGS 84 Manual First Edition, 1997). Whilst there remains a possibility of this proposal not forming the final standard, it will be possible to recompute CRC's at a later date on condition that the integrity has been assured up to that point.

6.4.2 Using a CRC for checking the correctness of data

Integrity cannot be added once it is lost. Thus the integrity assurance must be provided from origination to final application. In this respect the WGS 84 Programme must be responsible for origination of the CRC.

It is therefore proposed that when a data item is entered into electronic format it will need to be provided with the CRC and thereby provide the capability for verification on its transfer to the AIS. Then, and at each subsequent step, the receiver must confirm the data validity to provide assurance that it has not been corrupted while stored or during transmission.

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

- confirm the correctness against the original survey data.
- enter data under strict quality control procedures
- execute checks to verify data following entry.

For the CRC to be used for checking the correctness of data, the chosen CRC must be agreed between the system which produces the CRC and all systems which wish to receive the data. Furthermore, it 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 Implementation programme this smallest element will be a single point. *However a suitable set of data for a CRC check could be a procedure and ultimately, as the delivered set of data derived for the FMS which will not be changed, it could be a whole navigation data base.*

6.4.3 Relationship between security and integrity

It is important to note that the CRC process provides integrity, in that it allows a receiver of 'wrapped' data to determine whether there has been any corruption of the data, or the calculated CRCV, since the data was wrapped.

The CRC process does not provide security for the aeronautical data, in that it does not prevent the 'wrapped' data from becoming corrupted. Within the context of quality assurance, the CRC process can only detect inadvertent corruption of aeronautical data. The CRC process is a tool and not a single quality solution.

Safety is not compromised by a verification failure of a CRCV as procedures should be in place to ensure such data is removed from the distribution process.

6.4.4 Computer format differences

Ideally, once formed, the CRC would remain unchanged until its application in the FMS or RNAV system. However, data formats change during the process and a CRC is only valid for a particular format of data. For example data is held in a computer in a format dependant upon word length which is unlikely to be the same in the data base of the supplier and the RNAV/FMS. In addition one step of the processing is the conversion to ASCII to meet the ARINC 424 Format. Thus it will be necessary to recalculate the CRC at various steps in the management of data. This recalculation of CRC must be carried out under strict quality control if the CRC is to remain a valid indication of integrity.

6.4.5 Integrity and CRC length

CRC offers absolute assurance of error detection when there is only a single period of "burst error" within the stream of data which was subjected to the CRC, provided that the sub-string 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 the original and the corrupted data. Assuming an "even" mapping of valid data strings to CRC, the probability of 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.

Tab. 6-1 gives the length of 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 $3x10^{-8}$, it is therefore necessary to employ a 32 bit CRC.

INTEGRITY	***LENGTHS OF CRC***	
	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

Tab. 6-1. *Integrity and CRC length*

6.4.6 Example of CRC generation

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 x0 and of xn 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 is 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 zero.

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

Let D = Data, GP = Generator polynomial, Q = Quotient, R = Remainder

(suffixes O and R denote Transmitter and Receiver respectively).

Then: D = QO x GP + RO - for origination

D + R0 = QR x GP + RR - at receiver

Which can be represented as: D = QR x GP + RR + RO

It follows that: QO x GP x RO = QR x GP + RR + RO

Cancel GP and RO to give: QO = QR if RR = 0 (ie no errors)

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

For this example, the data = 11011001, the GP = 11001 and the CRC is 4 bits long. (One of the pre-requisites for the GP is that it is 1 bit longer than the CRC). Four zeros are appended to the data (the length of the CRC) and the data (with the four zeros added) is XORed with the GP. This operation yields a quotient and a remainder (the CRC).

Generation

An example for the generation of a CRC shows Fig. 6-2. The data = 11011001, the GP = 11001 and the CRC is 4 bits long. One of the pre-requisites for the GP is that it is 1 bit longer than the CRC. Four zeros are appended to the data (the length of the CRC) and the data (with the four zeros added) is XORed with the GP. This operation yields a quotient and a remainder (the CRC).

					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)

Fig. 6-2. *Data divided by GP to generate the CRC*

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

Fig. 6-3).

					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)

Fig. 6-3. *Data with CRC divided by GP to establish, whether corruption has occurred*

Receiving data

The CRC is needed to confirm receipt of data without corruption. The CRC, however, is no indicator of the quality of data. This has to be checked separately.

Appendices - Background Information

APPENDIX A

THE GLOBAL POSITIONING SYSTEM (GPS)

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. Developed 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.

System Organization

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

- 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.
- 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 20200 km above the earth and 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.
- The *User Segment*, comprising an unlimited number of receivers, which receive the satellite signals and calculate instantaneous position and other navigation information.

GPS SATELLITE SIGNAL STRUCTURE

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.

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 cesium 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

$$L1 = 1575.42 \text{ MHz} \quad (19 \text{ cm}) \qquad L2 = 1227.60 \text{ MHz} \ (24 \text{ cm})$$

These dual frequencies are essential for the elimination of the major source of error, i.e., ionospheric refraction. The pseudoranges that are derived from 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 $L2$.

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 29.31m is modulated on both carriers $L1$ and $L2$.

GPS SATELLITE MESSAGE

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

- satellite ephemerides,
- ionospheric modelling coefficients,
- status information,
- system time and satellite clock bias, and
- drift information.

The total message consisting of 1500 bits is transmitted in 30 seconds with a data rate of 50 Bit/sec. This message is subdivided into 5 subframes. One subframe is transmitted in 6 seconds and contains 10 words with 30 bits.

PSEUDORANGE AND CARRIER PHASE MEASUREMENTS

The following Fig. A-1 shows pseudorange 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 *Dt* called also *pseudorange*. Multiplying it by the velocity of light *c* (plus various corrections) results in a user-to-satellite distance.

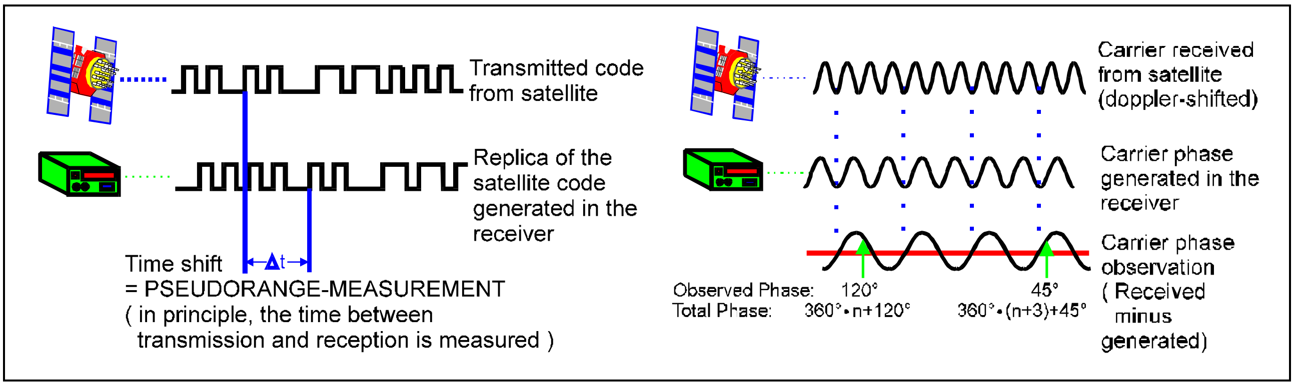


Fig. A-1. Pseudorange and carrier phase measurement

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.

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) is unknown, phase measurements are ambiguous: This ambiguity (= integer number of cycles) has to be determined in the processing.

SYSTEM ASSURANCE TECHNIQUES

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

Selective Availability (S/A)

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 pseudoranges (Dither-Process). This form of accuracy denial mainly affects single-receiver operations. When pseudoranges 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.

The second method of accuracy denial 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 translate to similar size position errors in the receiver.

Anti-Spoofing (A-S)

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 to create confusion and cause 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. A-S affects many of the high accuracy survey uses of the system.

GPS ABSOLUTE POSITIONING

Fig. A-2 shows that if a 3-D position is to be determined four pseudorange measurements to different satellites have to be measured.

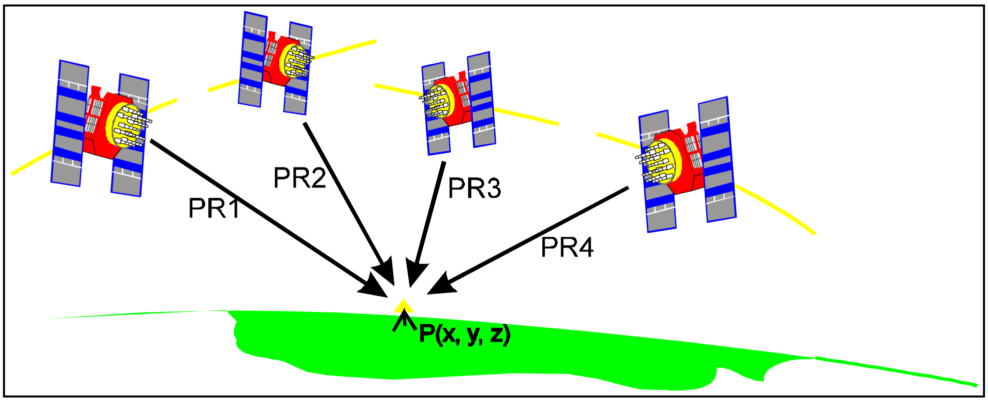


Fig. A-2. *Principle of GPS absolute positioning*

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.

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

$$(x1 - x)^2 + (y1 - y)^2 + (z1 - z)^2 + dT = (PR1)^2 \qquad \text{(A-1)}$$

$$(x2 - x)^2 + (y2 - y)^2 + (z2 - z)^2 + dT = (PR2)^2 \qquad \text{(A-2)}$$

$$(x3 - x)^2 + (y3 - y)^2 + (z3 - z)^2 + dT = (PR3)^2 \qquad \text{(A-3)}$$

$$(x4 - x)^2 + (y4 - y)^2 + (z4 - z)^2 + dT = (PR4)^2 \qquad \text{(A-4)}$$

GPS Single Point Absolute Positioning Accuracy

The civilian C/A code delivers a horizontal accuracy of 100 m (2dRMS) if S/A is on and 40 m (2dRMS) 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.

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.

POSITION ERRORS OF GPS

The following error sources of single point positioning are possible:

- Satellite orbit
- Satellite clock
- Satellite code: Selective Availability (S/A)
- Receiver: Resolution of the observation
- Receiver: Observation noise
- Antenna: Multipath effect
- Atmospheric refraction (ionosphere, troposphere)

DIFFERENTIAL GPS SURVEYING

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.

C/A Code Observations	**Receiver Noise**	**Differential Position Error (PDOP = 3)**
Pseudorange	0.2 . . . 5 m	0.6 . . . 15 m
Carrier Phase	0.2 . . . 2 mm	0.6 . . . 6 mm

Tab. A-1. *Observation noise and error propagation*

Tab. 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).

CHOICE OF GPS SURVEYING TECHNIQUE

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

Metre- and sub-metre accuracies:

Differential Phase-Smoothed Pseudorange Processing. Here the receiver costs are moderate (< 10 000 US $). Real-time surveying seems to be more robust than using pure phase measurements.

Centimetre accuracies:

Carrier Phase Based Approach. The necessary equipment and analysis software has a significantly higher price, mainly also 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 Tab. A-2.

GPS Surveying Modes and Accuracy

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

***Tab. A-2**. GPS surveying modes and accuracy*

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

Differential GPS Real-Time Positioning

Quite recently (mid-1994) differential GPS 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 a quality control in the field.

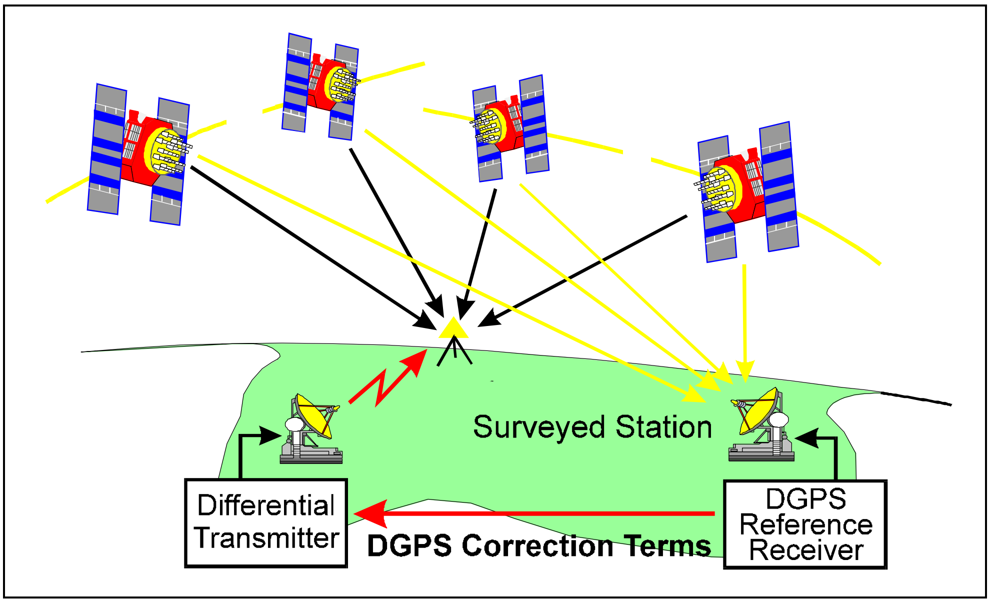


Fig. A-3. *Differential GPS real-time positioning*

Fig. A-3 demonstrates the principle of Differential GPS Real-Time Positioning:

- Positioning of a (roving) user relative to a reference station with known coordinates
- Determination of GPS PCP (Pseudorange and/or Carrier Phase) corrections at the reference station
- Transmission of the corrections to the mobile user by telemetry
- Quality and error control by monitor stations

DIFFERENTIAL CORRECTIONS

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

- Position or pseudorange corrections
- Carrier-Smoothed Pseudorange Corrections
- Carrier Phase Corrections.

Differential Pseudorange Corrections

A Differential Pseudorange Correction is the difference between observed and calculated (from known reference station coordinates and transmitted satellite ephemerides) pseudorange at the reference station.

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

Differential Carrier-Smoothed Pseudorange Corrections

The principle is the same as for the differential pseudorange corrections, but now the carrier phases are used to smooth the pseudoranges 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.
Possible real-time surveying accuracy: 0.6 ... 2 m

Differential Carrier Phase Corrections

Calculation and transmission of the corrections at known reference station. This is a new development with the following characteristics.

Advantages:
- Navigation in the centimetre range
- Smaller amount of telemetry data compared to raw data
- Reduction of time sensitivity
- Reduced computational burden for roving user

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
- Error detection is more difficult

Possible real-time surveying accuracy: 1 ... 5 cm

ACCURACY OF GPS

The following Fig. A-4 shows the GPS (absolute) and DGPS (relative) navigation and surveying accuracies achievable along with corresponding statistical distributions.

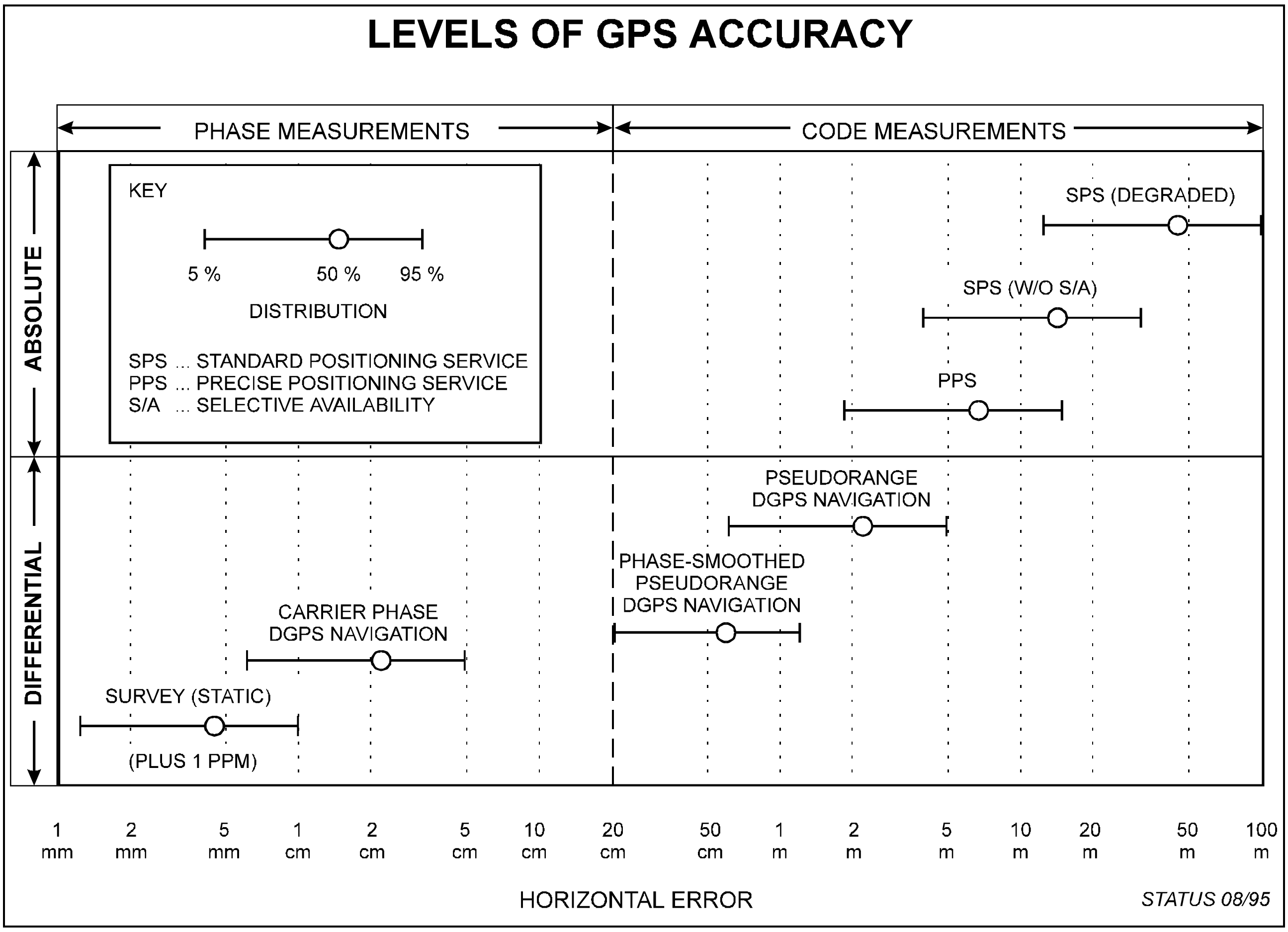


Fig. A-4. *Accuracy of GPS*

GPS DIFFERENTIAL POSITIONING TECHNIQUES

GPS differential positioning techniques have the following advantages and 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.
- GPS provides an all-weather 24 hour positioning capability.
- Various levels of accuracies are possible, depending on the available hardware and software.
- GPS is easy to use.
- Since certain signal obstruction by buildings, trees, etc. may occur, a certain amount of conventional surveying must be still be carried out.
- 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.

APPENDIX B

PRINCIPLES OF GEODESY

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.*
>
> *Friedrich R. Helmert (1880)*

This definition refers to Friedrich R. Helmert [8], one of the main founders of geodesy of the 19th century.

The following second definition represents a more up-to-date description of geodesy given by the Committee on Geodesy of the U. S. National Academy of Sciences in 1978.

- Establishment and maintenance of national and global three-dimensional geodetic networks
- Measurement and analyses of geodynamic phenomena (earth rotation, earth tides, crustal movements, etc.)
- Determination of the earth's gravity field

  - Items 1-3 include also changes with time

U.S. National Academy of Sciences
Committee on Geodesy (1978)

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

Why has geodesy to deal with the gravity field ?

First of all, every geodetic measurement is a function of the gravity field (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.

Secondly, in defining heights we have to use an equipotential surface of the gravity field as vertical reference (Where does water flow ?).

More on height definitions at the end of this Appendix

FIGURE OF THE EARTH AND REFERENCE SURFACES

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

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, ~ 800 BC). Pythagoras (~ 580-500 BC) and his school as well as Aristotle (384-322 BC) among others expressed themselves for the spherical shape.

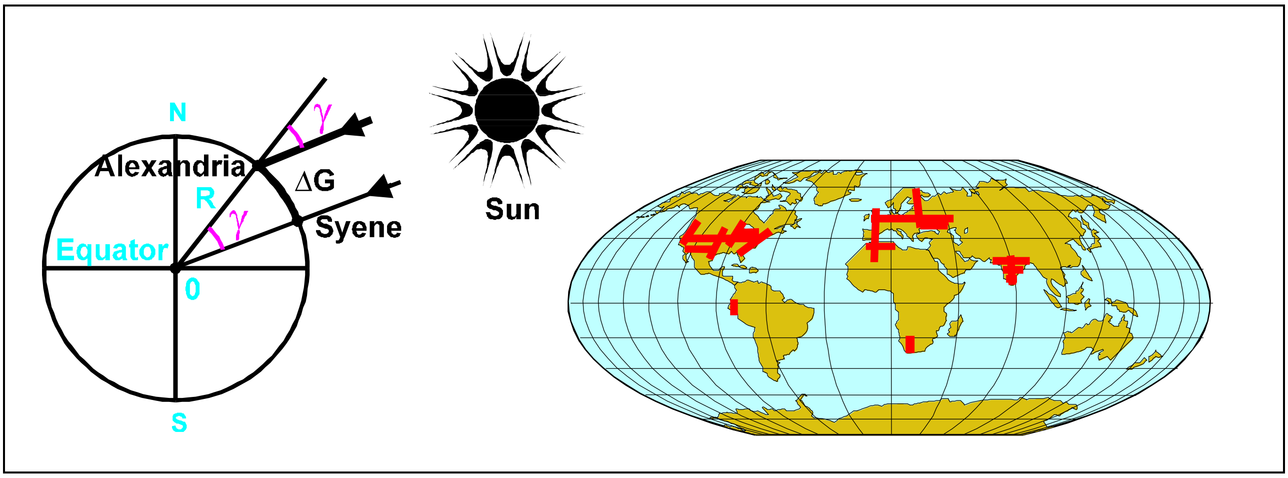


Fig. B-1. The Earth as a sphere, derived from arc measurements

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

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

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

The Earth as an Ellipsoid

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.

Flattening is defined by

$$f = (a - b)/a \qquad \text{(B-1)}$$

where a is the semimajor, and b is the semiminor axis of the ellipsoid.

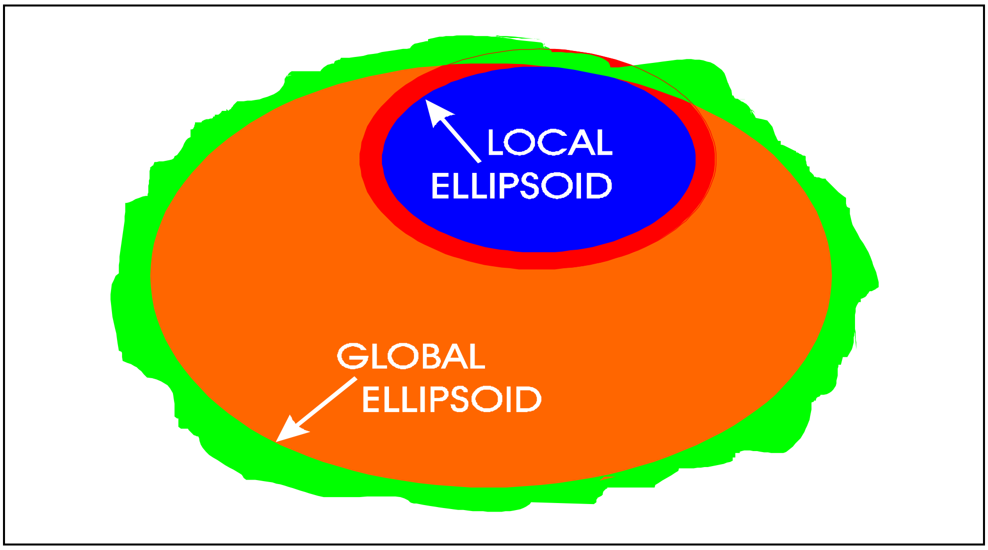


Fig. B-2. *Local ellipsoids are best-fitted to the specific country*

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

Tab. B-1 shows examples of the ellipsoidal parameters of various ellipsoids. Note that the East Europeans (former Soviet Union) based their horizontal coordinates on a triaxial ellipsoid (Krassowsky).

THE EARTH AS A GEOID

Laplace (1802), Gauss (1828), Bessel (1837) and others had already recognized that the assumption of an ellipsoidal earth model was not tenable when compared against 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

Fig. 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.

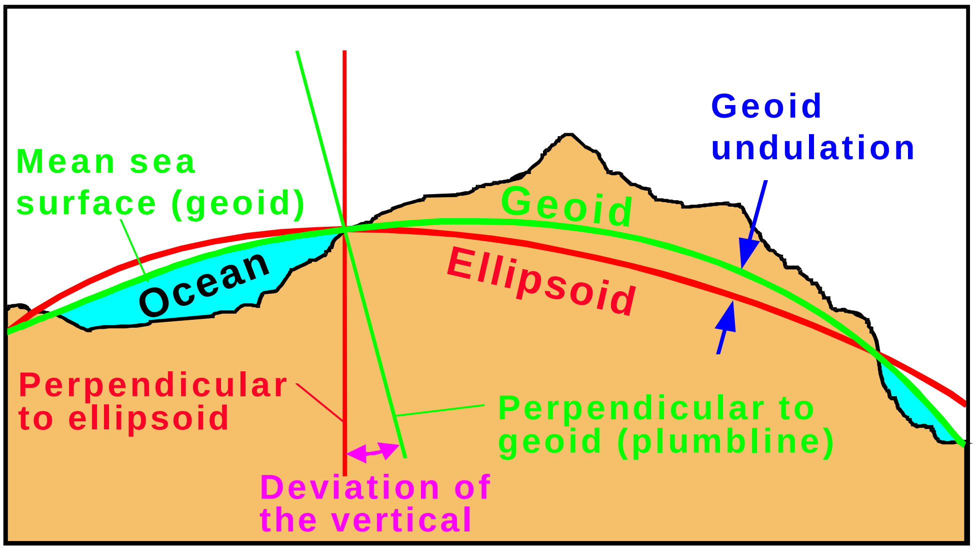


***Fig. B-3.** The Earth as a geoid*

The equipotential surface of the earth's gravity field which would coincide with the ocean surface, if the earth were undisturbed and without topography. *Listing (1873)*

Listing (1873) had given the name '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.

The determination of the geoid has been, for the last hundred years, a major goal of geodesy. Its importance increased recently by 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.

There are difficulties in defining a geoid: Sea-surface topography, sea-level rise (melting of the polar ice caps), density changes (earthquakes, etc.), ...

COORDINATE SYSTEMS AND REFERENCE ELLIPSOIDS

The *Geodetic Glossary* [7] gives the following general geodetic definitions:

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

Coordinate system
A set of rules for specifying how coordinates are to be assigned to points
→ origin, set of axes

Type of coordinate system
- Local
- Geocentric, earth-fixed
- Cartesian / Ellipsoidal

Local Coordinate System

In the past, national survey departments computed ellipsoids best-fitted to their country to provide the basis for mapping. Origin and orientation of coordinate system is 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 (Example: 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)

As a fundamental terrestrial coordinate system, one introduces an earth-fixed spatial Cartesian system (X, Y, Z) whose origin is the earth´s centre of mass S (geocentre, centre of mass including the mass of the atmosphere, see Fig. B-4. Earth-fixed spatial Cartesian system (X, Y, Z)). The Z-axis coincides with the *mean rotational axis of the earth (Polar motion, CIO Pole).*

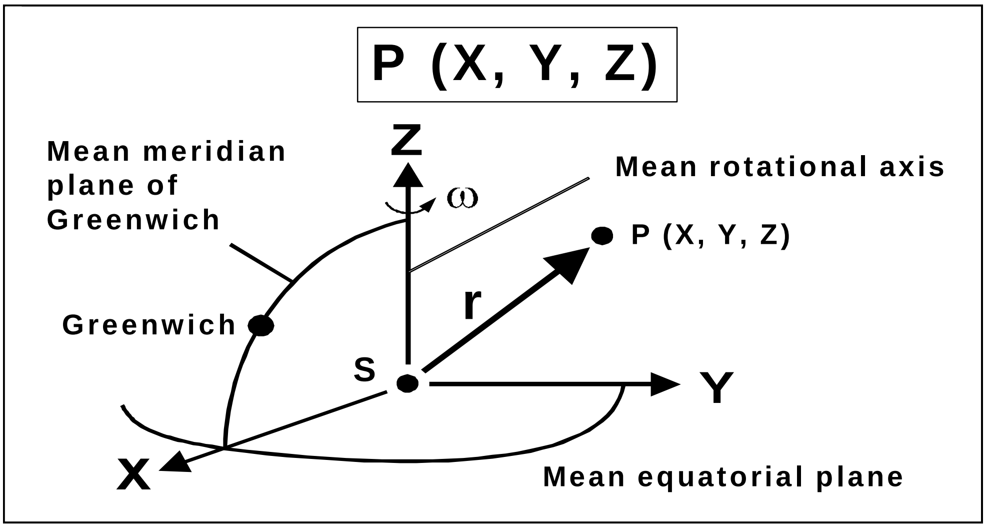


Fig. B-4. *Earth-fixed spatial Cartesian system (X, Y, Z)*

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 BIH (Bureau International de l´Heure) 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

As Fig. 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.

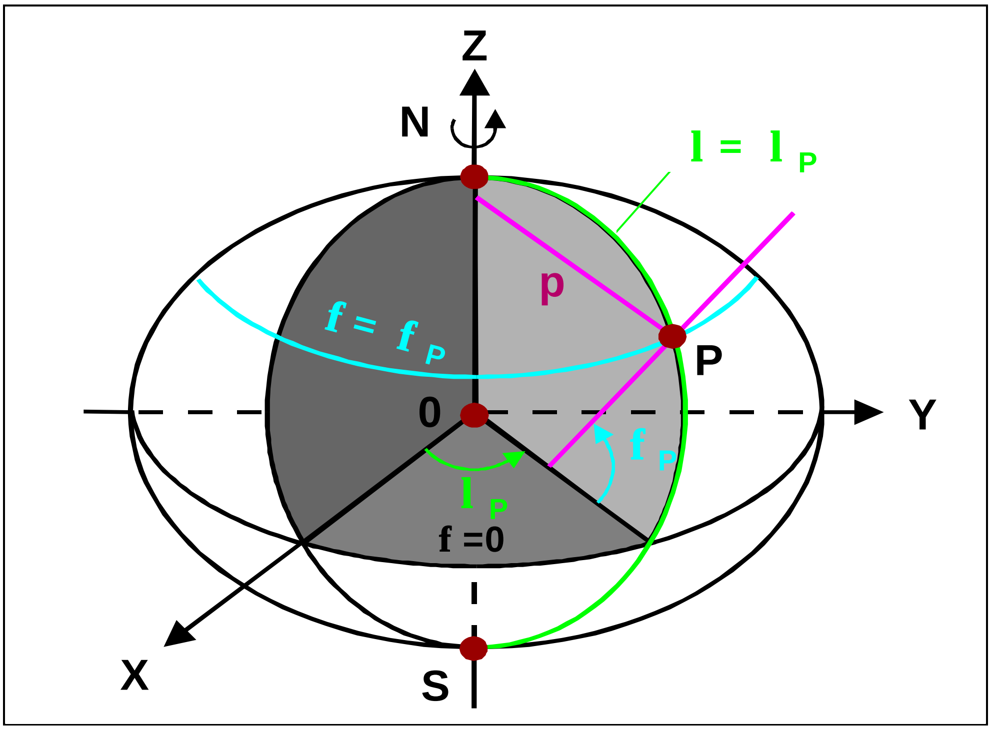


Fig. B-5. *Ellipsoidal geographic coordinates*

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 semimajor axis a and the semiminor axis b. Generally, b is replaced by a smaller parameter which is more suitable: the (geometrical) flattening f.

$$f = (a - b)/a \tag{B-2}$$

Further definitions:

- *Origin*
  Earth´s centre of mass
- *Geographic (geodetic) latitude* f
  Angle measured in the meridian plane between the equatorial (x, y)-plane and the surface normal at P
- *Geographic (geodetic) longitude* l
  Angle measured in the equatorial plane between the zero meridian (X-axis) and the meridian plane of P

Spatial Ellipsoidal Coordinate System

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 (Fig. B-6).

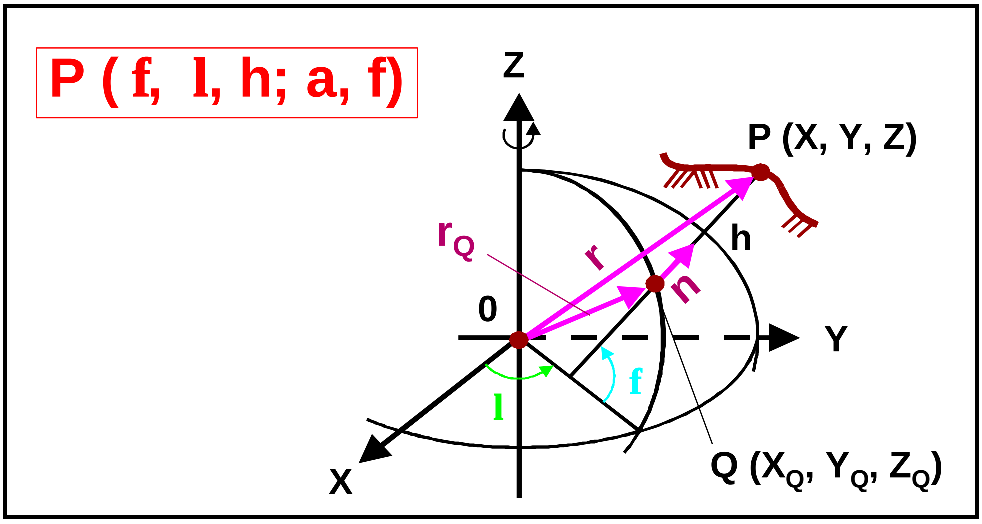


Fig. B-6. *Spatial ellipsoidal coordinate system*

The *spatial ellipsoidal coordinates f, l, 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).

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 also, it should possess a simple principle of formation.

In this respect, the *rotational ellipsoid*, already introduced as a geometric reference surface, is well suited. In addition to the semimajor 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.

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 $\overline{C}_{2,0}$ (2nd 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.*

Tab. 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.

REFERENCE ELLIPSOID NAME	*ID CODE*	*a (m), Da (m)*	*f^{-1}, $Df \times 10^{-4}$*
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 1972	WD	6378135, 2	298.26, 0.000312108
WGS 1984	WE	6378137, 0	298.257223563, 0

Tab. B-1. *Reference ellipsoid names and constants (WGS 84 minus local geodetic datum). See Ref. [4] for more detail*
**: As accepted by DMA*
***: Through adoption of a new yard to meter conversion factor in the reference country.*
Source: Ref. [4]

GEODETIC DATUM

Definitions

The terminology required to describe the geodetic datum problem is rather complex and has developed over more than 100 years. In order to avoid confusion and misunderstanding care will be taken to use the various terms precisely.

The following definitions are adopted by the international geodetic community:

Geodetic reference system (GRS)

- Conceptual idea of an earth-fixed Cartesian system (X, Y, Z)

Geodetic reference frame

- Practical realization of a geodetic reference system by observations

It is important to make a difference 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). In practical surveying we are only concerned with reference frames, but the underlying concepts of a specific reference frame are of fundamental importance.

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 ± to Z-axis
- Y-axis: Orthogonal

Local GRS

- Origin and orientation of axes is "arbitrary"

Geodetic datum

- Minimum set of parameters required to define location and orientation of the local system with respect to the global reference system/frame

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 Earth and the X-axis is contained in the mean meridian plane of Greenwich and is perpendicular to the Z-axis. The Y-axis is orthogonal to both the X- and Z-axis (right hand system).

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 one.

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

What is a Geodetic Datum ?

We have to distinguish between a *Cartesian datum* and an *ellipsoidal datum*.

A Cartesian datum is defined by a set of:

- 3 shifts: ΔX, ΔY, ΔZ
- 3 rotations: α, β, γ
- A scale factor parameter: μ

These 7 parameters are needed to relate two Cartesian 3-d reference frames.

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 has also 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 9 transformation parameters.

Rule of thumb:

Ellipsoidal Datum = Cartesian Datum + Shape of Earth Ellipsoid

TRANSFORMATIONS

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.

The nine parameters

- translation of the origin ΔX, ΔY, ΔZ,
- rotation angles ε_X, ε_Y, ε_Z,
- Scale factor μ,
- change in ellipsoidal semimajor 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.

More on Helmert’s Formula in Appendix E

THE HEIGHT PROBLEM

What is a "Height"

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 specific, it is an earth´s gravity potential surface. 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

The application of differential GPS satellite observations delivers

- Horizontal WGS 84 coordinates: ellipsoidal latitude ϕ and longitude λ,
- Vertical WGS 84 coordinates: ellipsoidal height h.

The ellipsoidal height does not have a physical meaning; it is a geometric quantity which does not indicate a level surface (i.e. it does not indicate the direction of 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

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

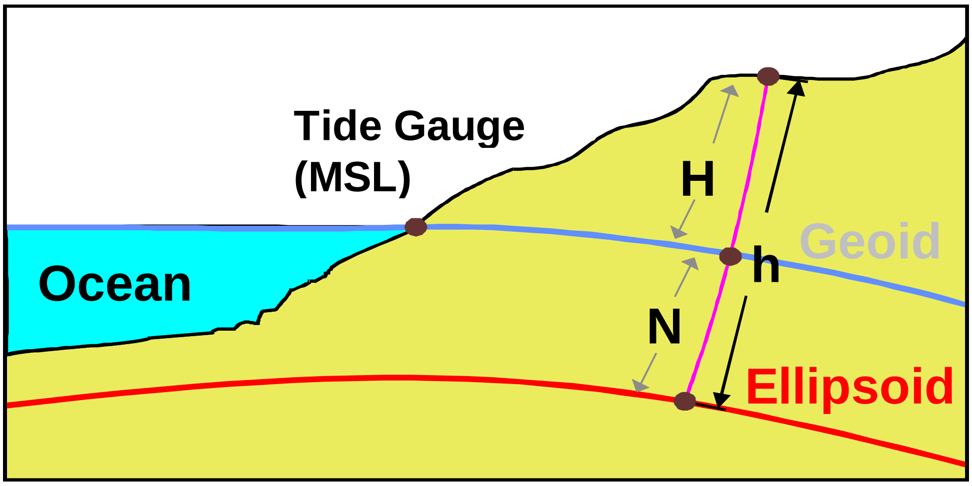


Fig. B-7. *The geoid as reference surface for heights*

The geoid is realized in practice by observing "mean sea level" (MSL) at tide gauges at the coasts over a certain time period. However, there are certain complications brought about by wind, salinity, currents, etc. producing 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.

Heights above the geoid are called "*orthometric heights H*". The relation between an ellipsoidal height h and H is given by

$$H = h - N \tag{B-3}$$

where N is the geoid height.

Vertical Datum Problem

WGS 84 is a 3-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.

In aviation, heights (flight level) are defined by atmospheric pressure. All aircraft are therefore equipped with baro-altimeters. For this reason the ICAO approach has been to initially use only WGS 84 geographical coordinates (ϕ, λ) and to exclude the geometric height (h) from consideration. However, because ICAO is considering the technical issues, surveyors are advised to measure and report the heights of navigation facilities, if the necessary field work and computation can be incorporated with any re-surveys of plan positions.

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 m!

Presently, there is world-wide effort to come up with an unified height system. It is hoped that this zero surface (namely the geoid) can be determined world-wide to an accuracy < 20 cm by using satellite altimetry.

The following Table shows vertical datum differences.

Vertical Datum Differences

Country/Reference Surface		**Difference to geoid**
Australia :	Mainland	- 68
	Tasmania	- 98
England		- 87
U.S.:	NGVD 29	- 26
	NAVD 88	- 72
	NAVD 88, East	- 38
Germany		4

Source: Rapp (1994)

Tab. B-2. *Reference surface differences with respect to the geoid Units: cm*

APPENDIX C

THE INTERNATIONAL TERRESTRIAL REFERENCE SYSTEM (ITRS)

DEFINITIONS

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 1-3 cm over distances up to about 5000 km. Global networks of up to 70 SLR- and up to 81 VLBI-stations are observing continuously, respectively were observed for limited periods. Since 1987 a new International Earth Rotation Service (IERS) is operating making use of SLR- and VLBI-results predominantly and producing every year a new global set of x, y, z-coordinates by combining various SLR- and VLBI-solutions [14].

The precise satellite laser ranging technique has led to a precise worldwide terrestrial coordinates 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).

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) and the zero meridian and the IERS Reference Meridian (IRM).

The maintenance of a datum at this level of accuracy requires constant monitoring of the rotation of the Earth, the motion of the pole 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 89, which means the computation of the ITRF coordinates at epoch 1989.0.

APPENDIX D

THE EUROPEAN TERRESTRIAL REFERENCE FRAME (ETRF)

HISTORY

Bearing in mind the necessity for a new very precise European Geodetic Reference System - called EUREF -, the International Association of Geodesy (IAG) decided in August 1987 to establish a new Sub-Commission to solve this task. One month later CERCO (Comité Européen des Responsables de la Cartographie Officielle) decided to establish a new Working Group VIII on GPS which was appointed to draw practical consequences resulting from the capabilities of GPS as a high precision positioning system for surveying and mapping. In October 1988 both bodies agreed that the definition and realization of a new European Reference System should be delt with as a common activity [14].

ETRF-89

Due to the fact that the combined SLR/VLBI-network is of highest achievable accuracy and by far the best global realization, the two European bodies decided to establish EUREF to be based on ITRS. Therefore 35 European SLR- and VLBI-positions being part of the ITRF-solution computed for 1989.0 were selected as the basic set of coordinates defining the realization of EUREF: ETRF, the European Terrestrial Reference Frame. So ETRF is a subset of the global solution ITRF at epoch 1989.0.

For SLR/VLBI *and ITRS see Appendix C*

Since the number of SLR- and VLBI-stations on the different national territories is by far not large enough, it was decided to use the precise differential GPS satellite technique to interpolate between those stations and to densify the network in Europe (EUREF Campaigns, started in 1989). This coordinate frame is now called ETRF-89.

Although ETRF-89 uses the GRS 80 ellipsoid it is identical, to the WGS 84 ellipsoid at the millimetre level.

Fig. D-1 shows the countries where, until the date above, EUREF campaigns were carried out to establish precise ETRF-89 coordinates (using GPS).

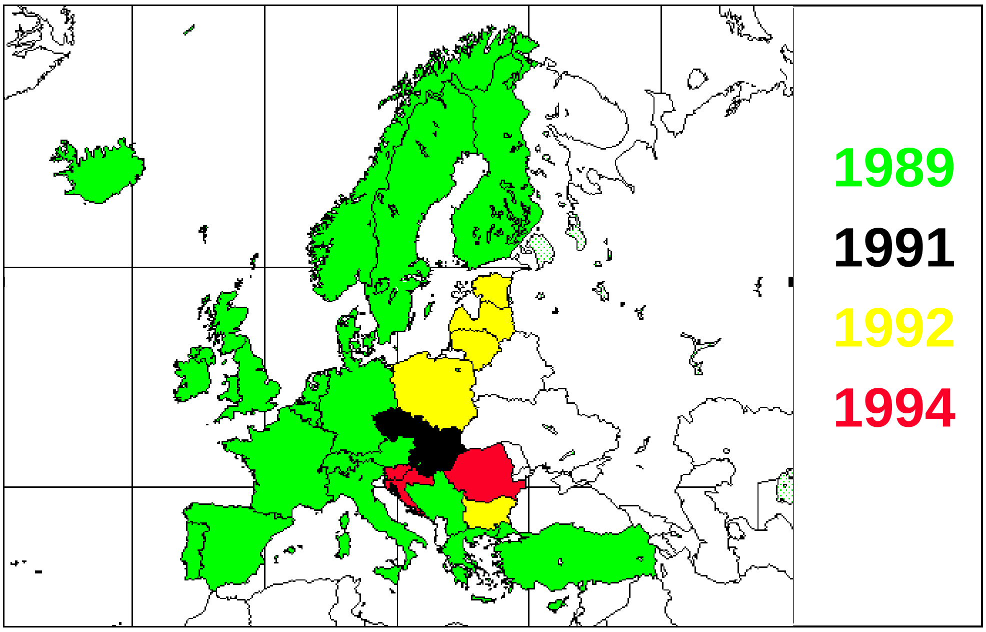


Fig. D-1. *Status ETRF-89/EUREF -01 Jan 1995-*

Source: Seeger (IfAG, Frankfurt)

APPENDIX E

DATUM TRANSFORMATION FORMULAS

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 semimajor axis a and flattening f).
- Find the geodetic latitude ϕ, longitude λ and ellipsoidal height h referring to the WGS 84 ellipsoid.

This procedure is illustrated in Fig. E-1.

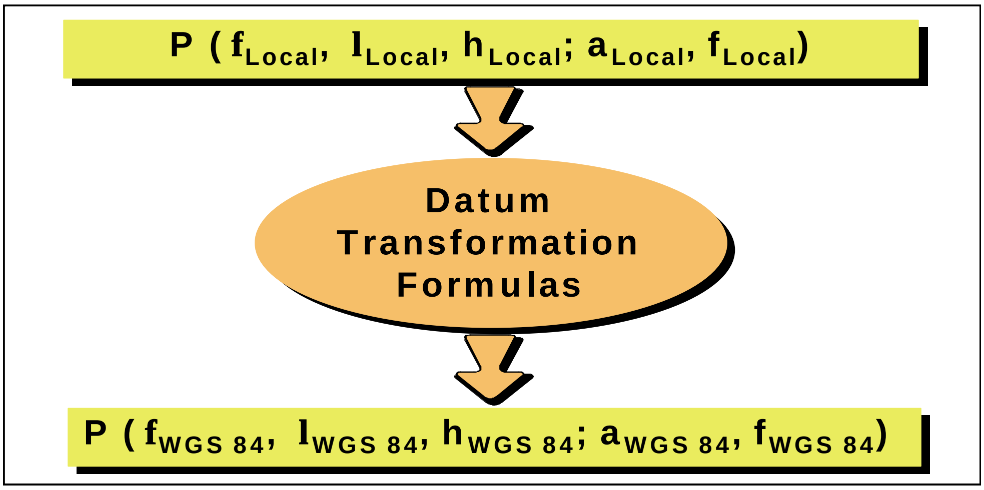


Fig. E-1. *General principle of a datum transformation*

The following three transformations are explained in more detail.

- *Helmert´s formula*
- *Standard Molodensky Formula*
- *The Multiple Regression Equation*.

The advantage of computational transformations over WGS 84 surveying is certainly 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 ... orthometric height and N ... height of the geoid. In general, only the orthometric height is known (and found also in maps). The geoid height has to be taken from a digital model (if available).

See APPENDIX B for more on height definitions

However, according to [2], 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 formulas. By assigning heights ranging from zero metres to 8000 m, it was concluded that the effect on both latitude and longitude was negligible (less than 15cm at 8000 m). Consequently, for a point of known latitude and longitude, but unknown (orthometric) height, an arbitrary height of zero metres could be assigned without significantly affecting the transformation.

HELMERT'S FORMULA

The application of Helmert´s formula requires a three step approach (*Step 1 through Step 3*), 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 right to *Step 2*.

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

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

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

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

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

ν : Radius of curvature in the prime vertical

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

See Tab. B-1 for a list of reference ellipsoids and parameters

a : Seemi – major axis of ellipsoid

e : Eccentricity of ellipsoid

$$e^2 = f(2 - f)$$

f : Flattening of ellipsoid

Step 2: Application of Helmert´s formula.
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.

See Tab. E-1 for a list of WGS 84 transformation parameters

$$\begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{WGS84} = \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{Local} + \underbrace{\begin{bmatrix} \mu & +\varepsilon_Z & -\varepsilon_Y \\ -\varepsilon_Z & \mu & +\varepsilon_X \\ +\varepsilon_Y & -\varepsilon_X & \mu \end{bmatrix}}_{\text{Rotation angles and scale factor}} \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{Local} + \underbrace{\begin{bmatrix} \Delta X \\ \Delta Y \\ \Delta Z \end{bmatrix}}_{\text{Shift of origin}}$$

Step 3: Afterwards, the back transformation from WGS 84 Cartesian coordinates into spatial ellipsoidal WGS 84 coordinates ϕ, λ, h is performed. It must be noted that the back transformation can only be carried out in an iterative manner. However, the development converges very fast due to $h << \nu$.

$$(\phi, \lambda, h)_{WGS84} \leftarrow (X, Y, Z)_{WGS84}$$

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

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

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

STANDARD MOLODENSKY FORMULA

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:

$$\phi_{WGS84} = \phi_{Local} + \Delta\phi$$
$$\lambda_{WGS84} = \lambda_{Local} + \Delta\lambda$$
$$h_{WGS84} = h_{Local} + \Delta h$$

with the Standard Molodensky Formula:

$$\begin{aligned}\Delta\phi'' = \{ & -\Delta X \sin\phi\cos\lambda - \Delta Y \sin\phi\sin\lambda + \Delta Z\cos\phi \\ & + \Delta a\,(\nu e^2 \sin\phi\cos\phi) \,/\, a + \Delta f\,[\,\rho\,(a/b) \\ & + \nu(b/a)]\sin\phi\cos\phi\} \cdot [(\rho + h)\sin 1'']^{-1}\end{aligned}$$

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

$$\begin{aligned}\Delta h = {} & \Delta X\cos\phi\cos\lambda + \Delta Y\cos\phi\sin\lambda + \Delta Z\sin\phi \\ & - \Delta a\,(a/\nu) + \Delta f\,(b/a)\,\nu\sin^2\phi\end{aligned}$$

$\Delta\phi$, $\Delta\lambda$, 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 (H: Orthometric height, N: Geoid height)	
ν	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	*See Tab. B-1 for a list of reference ellipsoids and parameters*
f	Flattening	
ΔX, ΔY, ΔZ	Shift of origin	*See Tab. E-1 for a list of WGS 84 transformation parameters*
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 e^2=f(2-f)	

The formulas should not to be used between 89^0 latitude and the pole !

MULTIPLE REGRESSION EQUATIONS

The development of Local Geodetic Datum to WGS 84 Datum transformation *Multiple Regression Equations* was initiated to obtain better fits over continental size land areas than could be achieved using the Standard Molodensky Formula with datum shifts Δx, Δy, Δz.

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 Ref. [4]

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

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

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

A_0, B_0 , C_0	...	constant
A_i (i=1 ... 9)	...	unknowns to be determined
B_i (i=1 ... 9)	...	unknowns to be determined
C_i (i=1 ... 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)

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

TRANSFORMATION PARAMETERS

Tab. E-1 gives a list of datum transformation parameters of existing national reference frames.

Reference Datum	***Translations (m)***			***Rotations (´´)***			***Scale m (ppm)***	***Comments***
	DX	***DY***	***DZ***	e_X	e_Y	e_Z		
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	.	.	.	.	
Nouv Trig de France	-168.0	-60.0	320.0	.	.	.	.	Greenwich Zero Meridian
Nouv Trig 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	
Hjorsey 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 M
Nouv Trig de Lux	-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	.	.	.	.	

Tab. E-1. *WGS 84 transformation parameters*

Note, that 1" in the rotation angle is approximately equal to 31 m on the earth-surface :

1" = 6 400 000 m x 3.141593 / (180 x 3600 ") = 31.03 m

[Also 1 Nautical Mile = 1852 metres (so 1 " = 30.48 m)]

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

APPENDIX F

SURVEYING AND PHOTOGRAMMETRIC METHODS

SURVEYING

Definition of Surveying

The following definition for surveying can be found in [7]:

- Deals primarily with so-called geometric measurements on the earth's surface
- Uses space methods like GPS for terrestrial positioning
- Computes derived quantities like coordinates, areas, etc.

Represents numerical data in graphical form like plans and maps

Conventional Surveying Techniques

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

For more information on ITRS and ETRF see Appendices C and D

Conventional surveying techniques determine

- *directions and angles using a theodolite* (accuracy of directions up to 0.01 mgon (or 0.1'' ; one gon is also called the grad which is 1/400 of a circumference). The technique is also called *triangulation.*
- *distances by EDM (electronic distance measurement) meters.* The accuracy derived can be up to 1 mm. The technique is also called *trilateration.*

Total Stations

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 whose coordinates are known and the telescope is pointed towards a target/reflector to measure distance, horizontal and vertical angles. These are recorded automatically either for immediate display (on-line field computation capabilities) or for post-processing.

Spirit Levelling

Fig. F-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 / √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 mentioned above 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.

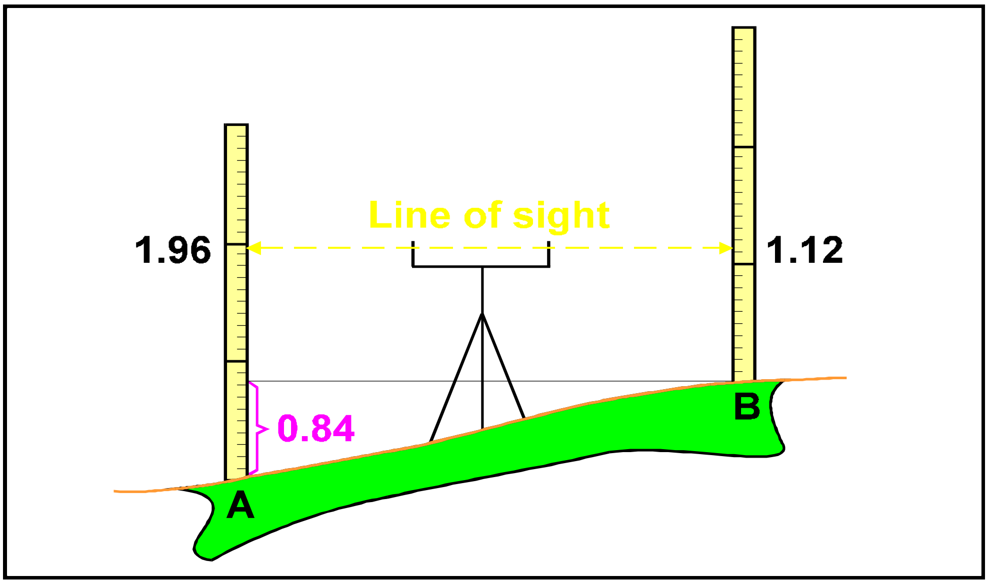


Fig. F-1. *Principle of spirit levelling*

Heights, which are derived using a trigonometric technique (measuring zenith or height angles), can be considered as "orthometric heights" (neglecting gravity data: accuracy in decimetre level).

Advantages/Disadvantages of Conventional Surveying

Field operations with modern total stations take a very short time and a surveyor with an assistant can, typically, survey up to 1000 points per day in favourable meteorological circumstances (theodolite, EDM instruments as well as spirit levelling instruments are less efficient). However, this technique has a number of drawbacks.

Firstly, 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 provided by triangulation points and heights 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 and third order, etc. In a densely surveyed country, 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 detail survey, by well known control survey techniques such as triangulation, trilateration and traversing.

GPS surveying and photogrammetric techniques are becoming progressively more efficient

The Height Problem Revisited

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

Orthometric heights of high accuracy can be only 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. Or in other words: To come up with high accuracy orthometric heights *orthometric corrections* have to be applied.

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

PRINCIPLES OF AEROPHOTOGRAMMETRY

The principle of aerophotogrammetry is briefly as follows:

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

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

Relative Orientation

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, the central perspective and take account of the tilts between images which are due to displacements of the aircraft (see Fig. F-2).

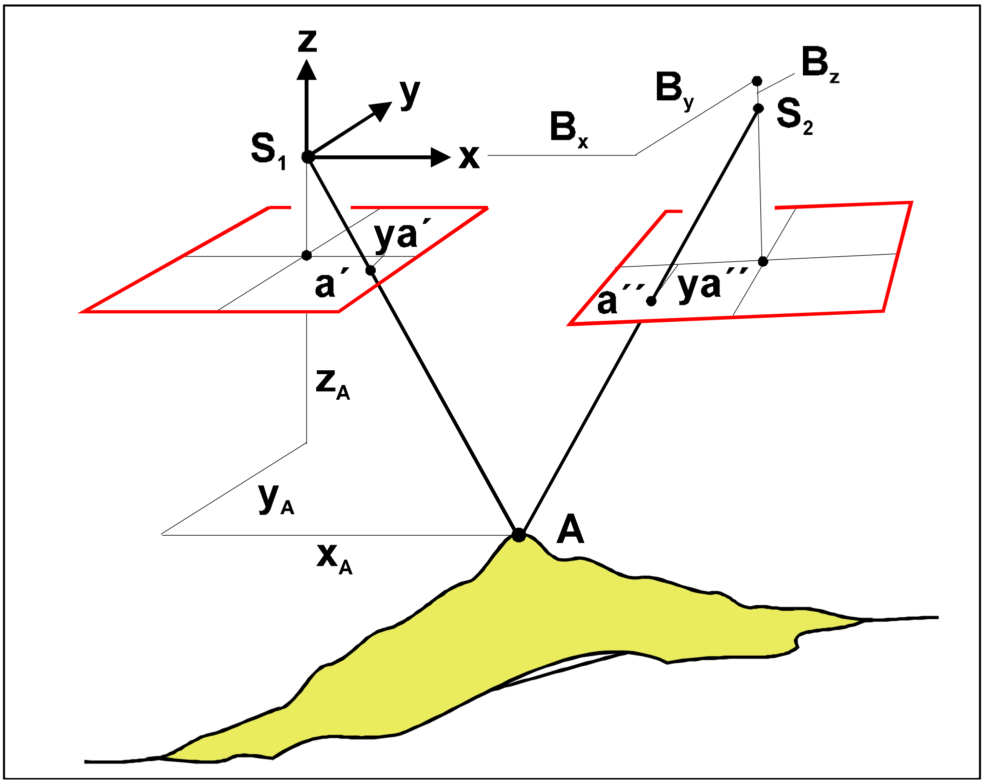


Fig. F-2. *Overlapping vertical photos taken by an aerial camera*

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

Absolute Orientation

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

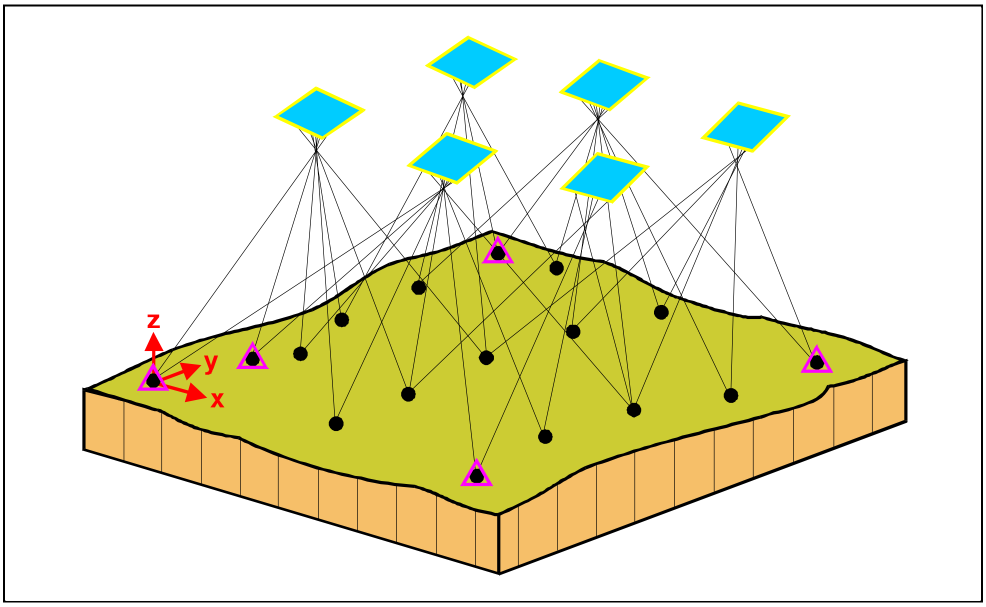


Fig. F-3. *Bundle block adjustment*

Minimization of Ground Control

Knowing that approximately 60-80 % 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 DGPS positioning (relative to a reference station). This requires a GPS receiver and antennae on the aircraft which is time-synchronized with the photogrammetric camera. In addition, the geometrical offset (eccentricity) between camera and GPS antenna has to be determined beforehand by measurement.

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

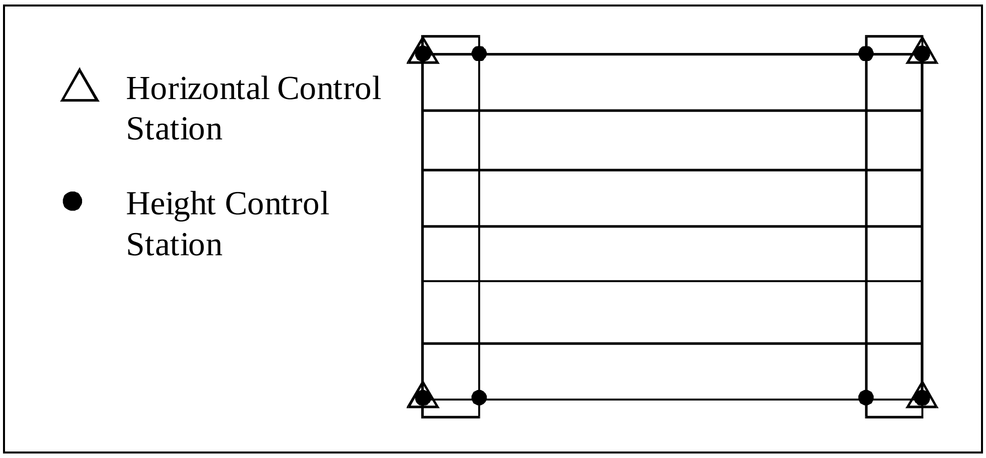


Fig. F-4. *Minimal amount of ground control by using DGPS*

Working Steps

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

- The parameters of the photo 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.
- The points to be coordinated have to be marked so that a unique identification in the aerial photos is possible.
- 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 approx. 50 m horizontal accuracy.
- The image coordinates are measured from the stereoscopic models establishment in photogrammetric instruments (e. g. an "Analytical Plotter").
- The final coordinates are derived computationally using a so-called photogrammetric block adjustment.

- Verification of the photogrammetrically-derived coordinates is done using selected field checks (GPS or conventional surveying).

Advantages/Disadvantages

The advantages of aerophotogrammetry are:

- A photogrammetric survey can cover a large area in one flight.

- The analogue photos which are taken for coordinate determination contain a lot of analogue information which might be useful for other tasks (interpretation, etc.)

The disadvantages may be:

- Due to the fact that the flights should be carried out when the vegetation is low (spring or late autumn) and the weather is clear, long waiting times my occur.

- It might not be very economical comparing it with other terrestrial techniques.

- Due to flight constraints and air traffic control certain restrictions may be present.

- The release of photos may need approval by governmental or military organizations.

APPENDIX G

MAP PROJECTIONS

The advances in information technology during the last two decades have given a boost to automatic cartography and hence, to digital mapping. A hard copy analogue map can now be digitized and transformed into a computer compatible data base, 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 the digitising of their national mapping, typical at scales from 1:1 000 for urban areas and up to 1: 10 000 for rural areas.

GENERAL PRINCIPLE

The general principle of a map projection is as follows:

General principle of map projections

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.

$$x = f_1(\phi, \lambda)$$
$$y = f_2(\phi, \lambda)$$

f_1 and f_2 can be a function of latitude and 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 by 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*.

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. Secondly, 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.

TYPE OF PROJECTIONS

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 (UTM, Gauss-Krüger, Lambert Conical, Stereographic Polar etc.). The surveyor mainly works in this plane coordinate system (x or Northing, y or Easting). The coordinates of surveyed objects, obtained by using for example EDM equipment and theodolites, are most easily computed by applying plane computation formulae.

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.

Projections can be classified as follows:

Mapping of the earth onto the plane of a

- *Azimuthal plane*
- *Tangent cone*
- *Tangent cylinder.*

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.

Properties of projections

Equidistance

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

Equivalence

An equal area map is one in which a . b = 1 (a, b are the axes of the ellipse of distortion).

Conformality

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

Geodetic map projections

Geodetic map projections differ from the cartographic ones as follows:

- The application is mainly with respect to large or medium scale maps.
- The reference ellipsoid is the one used by the national surveying agency.
- The graticule lines on the map represent *geodetic* coordinates. (In contrast: geographic coordinates on cartographic graticules.)
- Geodetic maps should represent the results of surveying (e. g. Northing, Easting).

Nowadays, mainly *conformal transverse cylindrical projections* are used (Gauss´ coordinates).

SPHERICAL PROJECTION FORMULAE

Transverse cylindric Mercator projection

It is a conformal projection. The meridian is the line of zero distortion, the principal scale is preserved along this meridian. At the equator the projection is equidistant, too. 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.

$$
\begin{aligned}
X &= R\ln(\sec\theta + \tan\theta) \\
Y &= R(\cos^{-1}(\tan\theta\cot(\lambda - \lambda_0) - \phi_0)) \\
\theta &= \sin^{-1}(\cos\phi\sin(\lambda - \lambda_0)) \\
R &= F_0\sqrt{\nu\rho} \\
\phi_0, \lambda_0 &= \text{latitude of origin point,} \\
& \quad \text{longitude of central meridian} \\
F_0 &= \text{Central meridian scale factor} \\
& \quad (F_0 = 0 \text{ for tangential})
\end{aligned}
$$

Stereographic polar

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 towards 90° east of the central meridian.

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

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

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

λ_0 = Longitude of central meridian

Stereographic oblique

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

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.

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

$$Y = r\sin\theta$$

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

$$r_0 = R\cot\phi_0$$

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

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

ϕ_0, λ_0 = Latitude of standard parallel

Longitude of central meridian

Lambert conical, two standard parallel

Similar to 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.

$$
\begin{aligned}
X &= r_0 - r\cos\theta \\
Y &= r\sin\theta \\
\theta &= (\lambda - \lambda_0)\sin\phi_0 \\
r_0 &= \frac{R\cos\phi_1}{\sin\phi_0}\left(\frac{\tan(45-0.5\phi_0)}{\tan(45-0.5\phi_1)}\right)^{\sin\phi_0} \\
r &= \frac{R\cos\phi_1}{\sin\phi_0}\left(\frac{\tan(45-0.5\phi)}{\tan(45-0.5\phi_1)}\right)^{\sin\phi_0} \\
\sin\phi_0 &= \frac{\ln\cos\phi_1 - \ln\cos\phi_2}{\ln\tan(45-0.5\phi_1) - \ln\tan(45-0.5\phi_2)}
\end{aligned}
$$

ELLIPSOIDAL PROJECTION FORMULAE

Note: For the ellipsoidal formulae the descriptions and definitions of cartesian axes are the same as for the spherical formulae. As a result only the formulae are given with no further comment.

Stereographic polar

$$
\begin{aligned}
X &= -r\cos\theta \\
Y &= r\sin\theta \\
\theta &= \lambda - \lambda_0 \\
r &= 2a(1+e)^{-1/2(1+e)}(1-e)^{-1/2(1-e)}\left(\frac{\cos\phi}{1+\sin\phi}\right)\left(\frac{1+e\sin\phi}{1-e\sin\phi}\right)^{1/2e} \\
\lambda_0 &= \text{longitude of central meridian} \\
a &= \text{semi-major axis of ellipsoid} \\
e &= \text{eccentricity of ellipsoid}
\end{aligned}
$$

Stereographic oblique

$$X = 2\nu_0\left(\frac{1+e}{1-e}\right)^{-1/2}\left(\frac{\sin\phi\cos\phi_0 - \cos\phi\sin\phi_0\cos(\lambda-\lambda_0)}{1+\sinh}\right)\left(\frac{1+e\sinh}{1-e\sinh}\right)^{-1/2}$$

$$Y = 2\nu_0\left(\frac{1+e}{1-e}\right)^{-1/2}\left(\frac{\cos\phi\sin(\lambda-\lambda_0)}{1+\sinh}\right)\left(\frac{1+e\sinh}{1-e\sinh}\right)^{-1/2}$$

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

$$\upsilon_0 = \left(\frac{a}{(1-e^2\sin^2\phi_0)}\right)^{1/2}$$

ϕ_0, λ_0 = Latitude of tangential pole
Longitude of tangential pole

Lambert conical, one standard parallel

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

$$Y = r\sin\theta$$

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

$$r_0 = \upsilon_0\cot\phi_0$$

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

$$\nu_0 = \frac{a}{(1-e^2\sin^2\phi_0)^{1/2}}$$

ϕ_0, λ_0 = Latitude of standard parellel
Longitude of the central meridian

Lambert conical, two standard parallel

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

$$Y = r\sin\theta$$

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

$$r_0 = \nu_2 \frac{\cos\phi_2}{\sin\phi_0}\left(\frac{\tan(45-0.5\phi_0)\left(\dfrac{1+e\sin\phi_0}{1-e\sin\phi_0}\right)^{1/2e}}{\tan(45-0.5\phi_2)\left(\dfrac{1+e\sin\phi_2}{1-e\sin\phi_2}\right)^{1/2e}}\right)^{\sin\phi_0}$$

$$r = \nu_2 \frac{\cos\phi_2}{\sin\phi_0}\left(\frac{\tan(45-0.5\phi)\left(\dfrac{1+e\sin\phi}{1-e\sin\phi}\right)^{1/2e}}{\tan(45-0.5\phi_0)\left(\dfrac{1+e\sin\phi_0}{1-e\sin\phi_0}\right)^{1/2e}}\right)^{\sin\phi_0}$$

$$\sin\phi_0 = \frac{\ln\nu_1\cos\phi_1 - \ln\nu_2\cos\phi_2}{\ln\left(\tan(45-0.5\phi_1)\left(\dfrac{1+e\sin\phi_1}{1-e\sin\phi_1}\right)^{1/2e}\right) - \ln\left(\tan(45-0.5\phi_2)\left(\dfrac{1+e\sin\phi_2}{1-e\sin\phi_2}\right)^{1/2e}\right)}$$

$$\nu_1 = \frac{a}{(1-e^2\sin^2\phi_1)^{1/2}}$$

$$\nu_2 = \frac{a}{(1-e^2\sin^2\phi_2)^{1/2}}$$

Transverse Mercator projection

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

$$
\begin{aligned}
X &= M + (\lambda-\lambda_0)^2 A + (\lambda-\lambda_0)^4 B + (\lambda-\lambda_0)^6 C \\
Y &= (\lambda-\lambda_0) D + (\lambda-\lambda_0)^3 E + (\lambda-\lambda_0)^5 F \\
M &= b((1 + n + \frac{5}{4}n^2 + \frac{5}{4}n^3)(\phi-\phi_0) \\
&\quad - (3n + 3n^2 + \frac{21}{8}n^3)\sin(\phi-\phi_0)\cos(\phi+\phi_0) \\
&\quad + (\frac{15}{8}n^2 + \frac{15}{8}n^3)\sin 2(\phi-\phi_0)\cos 2(\phi+\phi_0) \\
&\quad - \frac{25}{24}n^3 \sin 3(\phi-\phi_0)\cos 3(\phi+\phi_0) \\
A &= \frac{\nu}{2}\sin\phi\cos\phi \\
B &= \frac{\nu}{24}\sin\phi\cos^3\phi(5 - \tan^2\phi + 9\eta^2) \\
C &= \frac{\nu}{720}\sin\phi\cos^5\phi(61 - 58\tan^3\phi + \tan^4\phi) \\
D &= \nu\cos\phi \\
E &= \frac{\nu}{6}\cos^3\phi(\frac{\nu}{\rho} - \tan^2\phi) \\
E &= \frac{\nu}{6}\cos^3\phi(\frac{\nu}{\rho} - \tan^2\phi) \\
F &= \frac{\nu}{120}\cos^5\phi(5 - 18\tan^2\phi + \tan^4\phi + 14\eta^2 - 58\tan^2\phi\eta^2) \\
\nu &= \frac{a}{(1 - e^2\sin^2\phi)^{1/2}} \\
\rho &= \frac{a(1-e^2)}{(1 - e^2\sin\phi)^{3/2}} \\
\eta &= \frac{a-b}{a+b} \\
b &= a(1-f) \\
\eta^2 &= \frac{\nu}{\rho} - 1
\end{aligned}
$$

The central meridian scale factor F_0 is applied by multiplying the semimajor axis a by F_0 before calculating any other quantities.

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. 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 3^0 width (in longitude) are established, the central meridian lying in the middle of each zone. The meridional strip systems have the following northings and eastings:

Northing Distance from 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 following transformation example shows the way for determining the WGS 84 coordinates of the tower of the town hall of Berlin, which is given in Gauss-Krüger coordinates. The transformation equations are taken from [9]. Firstly, the tower coordinates have to be projected onto the Bessel ellipsoid by taking the central meridian $\lambda_0 = 12°$. Secondly, the geographical coordinates ϕ and λ with respect to the Bessel ellipsoid have to be transformed to the WGS 84 ellipsoid.

The tower of the town hall of Berlin has the Gauss-Krüger coordinates:

Easting y = 45 95 696.17 m, Northing x =5821529.31 m

The parameters for the Bessel ellipsoid can be taken from Tab. B-1:

$a = 6377397.155$, $b = a\,(1-f) = 6356078.963$
$n = (a-b)/(a+b)$ $e'^2 = (a^2 - b^2)/b^2$

The latitude of the footprint ϕ_f is given by:

$$\phi_f = \frac{x}{\bar{\alpha}} + \bar{\beta}\sin\frac{2x}{\bar{\alpha}} + \bar{\gamma}\sin\frac{4x}{\bar{\alpha}} + \bar{\delta}\sin\frac{6x}{\bar{\alpha}} + \bar{\varepsilon}\sin\frac{8x}{\bar{\alpha}} + \ldots$$

with

$$\bar{\alpha} = \frac{a+b}{2}\left(1 + \frac{1}{4}n^2 + \frac{1}{64}n^4 + \ldots\right) \quad \bar{\gamma} = \frac{21}{16}n^2 - \frac{55}{32}n^4 + \ldots$$

$$\bar{\beta} = \frac{3}{2}n - \frac{27}{32}n^3 + \frac{269}{512}n^5 + \ldots \qquad \bar{\delta} = \frac{151}{96}n^3 - \frac{417}{128}n^5 + \ldots \;;\; \bar{\varepsilon} = \frac{1097}{512}n^4 + \ldots$$

Radius of curvature in the prime vertical N_f:

$$N_f = \frac{a^2}{b\sqrt{1+e'^2\cos^2\phi_f}} = \frac{a^2}{b\sqrt{1+\eta^2{}_f}}; \quad t_f = \tan\phi_f$$

Using the projection formulas of [9]

$$\phi = \phi_f + \frac{t_f}{2N_f^2}(-1-\eta_f^2)y^2 + \frac{t_f}{24N_f^4}(5+3t_f^2+6\eta_f^2-6t_f^2\eta_f^2-3\eta_f^4-9t_f^2\eta_f^4)y^4$$

$$+\frac{t_f}{720N_f^6}(-61-90t_f^2-45t_f^4-107\eta_f^2+162t_f^2\eta_f^2+45t_f^4\eta_f^2)y^6+\ldots$$

$$\lambda = \lambda_0 + \frac{1}{N_f\cos\phi_f}y + \frac{1}{6N_f^3\cos\phi_f}(-1-2t_f^2-\eta_f^2)y^3$$

$$+\frac{1}{120N_f^5\cos\phi_f}(5+28t_f^2+24t_f^2+6\eta_f^2+8t_f^2\eta_f^2)y^5+\ldots$$

yields to the geographical coordinates, referring to the Bessel ellipsoid:

$\lambda_{Bessel} = 13^0 24'36.01''$ $\phi_{Bessel} = 52^0 31'11.65''$

In order to get WGS 84 coordinates, a datum transformation from the Potsdam datum to the WGS 84 system has to be performed. The approximated transformation parameteres can be found in Tab. E-1 and in the *DATUM* software. They are:

$\Delta X = 587.0$ m; $\Delta Y = 16.0$ m; $\Delta Z = 393.0$ m

Using the *DATUM* software gives the WGS 84 coordinates of the tower of the town hall of Berlin:

$\lambda_{WGS84} = 13^0 24'29.62''$ $\phi_{WGS84} = 52^0 31'06.71''$

UTM-SYSTEM

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

- The projection is the Gauss-Krüger version of the Transverse Mercator intended to provide world coverage between the latitudes 84^0N / 80^0S.
- The reference ellipsoid is the International 1924, the unit of measure is the International Metre.
- Each zone is 6^0 of longitude in width; the first zone has its western edge on the meridian 180^0 and the zones proceed eastwards to zone 60 which has its eastern edge at 180^0 longitude. The central meridian of each zone is therefore 177^0 in zone 1, 171^0 in zone 2 and 165^0 in zone 3.

- 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 pole-wards as 84^0 N and 80^0 S. Initially these limits were set at 80^0 N and 80^0 S.
- The eastings of the origin of each zone is assigned the value of 500000 m.
- The scale factor on the central meridian is 0.9996.
- The UTM employs five different figures, for specific areas.

APPENDIX H

SAMPLE QUESTIONNAIRE

WGS 84 Implementation Programme

Survey Inventory

Questionnaire

PART I EN-ROUTE NAVAIDS

Please complete in black in block capitals
Make selections as indicated [x]

State	
Civil Aviation Administration	
Address	
Contact Name	
Tel	
Fax	

Please complete and return to:

Navigation Aid Survey Inventory

Questionnaire

PART I EN-ROUTE NAVAIDS

(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 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) was generally used ?

	DME	VOR	DME/VOR	NDB	VORTAC	TACAN
>1/5000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/5000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/10000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/20000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/25000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/50000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/100000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1/250000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
<1/250000	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]

---

6. To what accuracy have the coordinates been determined ?

Note: This may differ from the resolution to which they are 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 were extracted from a map, is it known on which datum the map is based ?

[ ] Yes [ ] No

---

8. Was this information (at 7) recorded as part of the survey ?

[ ] Yes [ ] No

---

9. If instrument surveys were performed,

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

[ ] Yes [ ] No

b) Was 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 level of staff are 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. Were field inspections undertaken to verify the location of the navigation aids and are they part of an on-going programme for inspection ?

[ ] Yes [ ] No

14. Are such inspections, or similar inspections part of an on-going 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 inconsistency ?

[ ] Yes [ ] No

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

[ ] Yes [ ] No

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

[ ] 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 their 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 (ie date on which coordinates were determined) ?

[ ] Yes [ ] No

(iv) Method of survey

[ ] Yes [ ] No

(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 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 (eg VOR/DME) do the coordinates relate to the distance measuring element ?

[ ] Yes [ ] No

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