FIG GUIDE

FIG PUBLICATION
NO 64

Reference Frames in Practice Manual

Commission 5 Working Group 5.2 Reference Frames

May 2014

INTERNATIONAL FEDERATION
OF SURVEYORS (FIG)

Reference Frames in Practice Manual

Commission 5 Working Group 5.2 Reference Frames

Editor: Graeme Blick

INTERNATIONAL FEDERATION OF SURVEYORS (FIG)

This Technical Manual is produced by FIG Commission 5 and has been contributed to by a number of Technical experts. The objective of the Manual is to provide a brief introduction to the use of Reference Frames in Practice. It is arranged as a series of short fact sheets that can be easily added to and updated.

Any feedback or suggestions on content should be forward to FIG Commission 5 Chair – see http://www.fig.net/commission5/

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FOREWORD

The International Federation of Surveyors (FIG) Commission 5 is responsible for assisting practising surveyors in FIG member associations to apply Positioning and Measurement technologies efficiently and effectively in their day-to-day survey activities. One of the most significant technologies to emerge in recent decades has been Global Navigation Satellite Systems (GNSS). The rise of such a global technology has highlighted the need for countries to move from locally defined geodetic datums to more global datums based on the International Terrestrial Reference Frame. This FIG publication is a response by Commission 5 to this trend by bringing together a series of fact sheets to better inform surveyors about some of the key issues they need to consider as they realign and upgrade their professional knowledgebase.

During my time in Commission 5 as a Working Group Chair and then as Commission Chair, we placed special emphasis on improving the availability of factual and accessible information about reference frame matters, especially for surveyors and decision makers in developing countries. That work was extended and improved by the Chairs of Commission 5 who followed me, Rudolf Staiger and Mikael Lilje. A special feature of the Commission's work in recent years was a series of workshops on reference frame matters that were attached to FIG and allied events in developing countries.

FIG would like to acknowledge the partnership under our MoU with the UN Office for Outer Space Affairs, which saw the UN Office fund developing country delegates to attend several of the international workshops that form the basis of this publication. That interaction was very important in identifying the specific topics that this publication should cover and help surveyors to better understand.

An extremely important aspect of this publication is as a concrete demonstration of the value of the increased cooperation in recent years between FIG and its sister association, the International Association of Geodesy (IAG). The international workshops and resulting fact sheets have seen a very close collaboration between internationally recognised experts from both FIG and IAG and it is hoped that such collaboration will continue to grow and deepen in the future. FIG would like to especially acknowledge the role played by IAG President Chris Rizos as a driving force behind IAG's commitment to our increased cooperation.

In closing FIG and its Member Associations are very grateful to the experts from both FIG and IAG who generously volunteered their time and effort to support the international workshops and especially to those who went on to author the fact sheets that make up this important publication. Special mention goes to Commission 5 Vice-Chair Graeme Blick, who coordinated the development of this publication under the leadership of Commission Chair Mikael Lilje and with the very able administration of Commission Vice-Chair Rob Sarib.

Matt Higgins
Former Chair FIG Commission 5 and FIG Vice President
FIG Honorary Member

1 INTRODUCTION

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This manual addresses technical issues surrounding reference frames, presenting formulae when appropriate. It is arranged as a series of short fact sheets that can be easily added to and updated. The objective of the manual is to provide a brief introduction to the use of Reference Frames in Practice. It is intended for surveyors but does assume some knowledge of the topic. It contains a number of technical terms that may not be detailed and lists references where additional information may be found.

Background

Since the dawn of civilisation, people have found it necessary to measure and map their domain (Fig. 1). During very early times this concern was limited to the immediate vicinity of their home; later it expanded to the distance of markets; and finally, with the development of means of transportation and communication people became interested in the whole world.

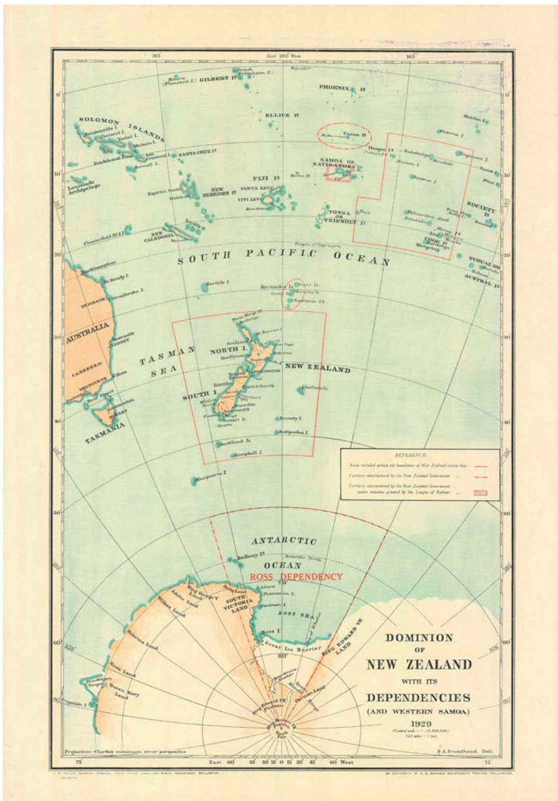

Figure 1: *Early map of SW Pacific.*

Much of this early "world interest" was evidenced by speculation concerning the size, shape, and composition of the Earth.

Geodesy is the science of measuring the shape and size of the Earth and precisely locating points on its surface. As our society and economy becomes increasingly dependent on complex technologies and the management of the space we live in, the need for precise positioning and consistent, reliable spatial data has intensified.

As we move to a world where new technologies allow us to rapidly determine the accurate position of features and points, we are developing the concept of everything 'geodetic'. That is the development of a seamless geodetic cadastre and all spatial datasets in terms of a common geodetic system.

For many countries subject to the effects of ground movements due to events such as earthquakes, volcanic activity or plate tectonics, the ability to survey and record these movements to maintain accuracy of the geodetic system is an important task. A country's geodetic system provides the network of permanent ground reference points and the associated intellectual and positional data that enables it to ensure all data concerning land, resources, and location is managed in a systematic and orderly manner.

Fundamental to any geodetic system is the spatial reference frame upon which it is based. Historically these were locally or regionally based, but as we have transitioned to the use of globally based satellite positioning systems our reference frames have become much more global in nature.

A spatial reference frame allows a location to be unambiguously identified through a set of coordinates (usually latitude and longitude or northing and easting).

Datums and Projections

A **geodetic datum** is a curved reference surface that is used to express the positions of features consistently. Geodetic datums are usually classified into two categories: local and geodetic.

A **Local Geodetic Datum** is a datum which best approximates the size and shape of a particular part of the Earth's sea-level surface (Fig. 2). It is defined by specifying a reference ellipsoid, the position (latitude and longitude) of an initial station and an azimuth from that station. Invariably, the centre of its ellipsoid will not coincide with the Earth's centre of mass. Until very recently, most national geodetic systems were based on local geodetic datums.

A **Geocentric Datum** is a datum which best approximates the size and shape of the Earth as a whole. The centre of its ellipsoid coincides with the Earth's centre of mass (Fig. 3). Geocentric datums do not seek to be a good approximation to any single part of the Earth but on average they are a good fit.

Global Navigation Satellite Systems (GNSS) utilise geocentric datums to express their positions because of their global extent. Multiple GNSS are now fully operational or being developed such as GPS, GLONASS, GALILEO, and BEIDOU and each uses a slightly different geocentric datum.

The World Geodetic System 1984 (used by GPS) is an example of a geocentric datum.

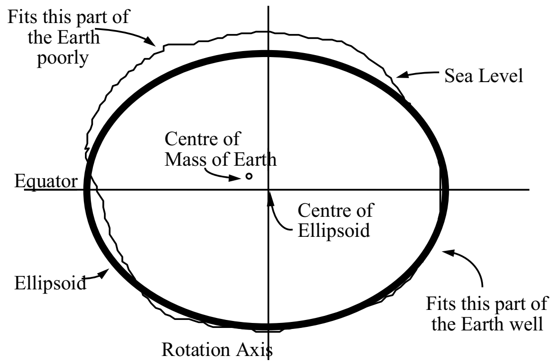

Figure 2: *Local datum with best fit ellipsoid.*

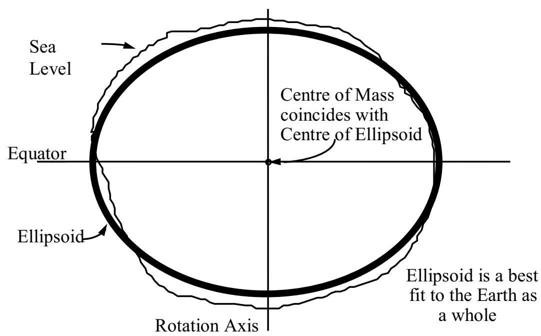

Figure 3: *Geocentric datum with ellipsoid that is a best fit to the world.*

Mean sea level is widely used as the **reference surface for the measurement of height**. The contours on a map will usually show height above mean sea level. However, heights in terms of a geodetic datum will be in relation to an ellipsoid.

Another type of coordinate system is provided by **map projections**. A map projection enables the curved surface of the ellipsoid to be represented on a flat sheet of paper (in other words, a map). Projections are used to define local coordinate systems on which calculations of distance, direction, and area are less complex than the equivalent computations on the ellipsoid.

Many projections can be defined in terms of a particular geodetic datum, but each projection can only be linked to a single geodetic datum.

The projection process results in the map's spatial representation being distorted. Imagine the stretching and tearing that you would have to do to a basketball to make its curved surface lie flat on the ground. The magnitude of the distortion can be calculated, allowing corrections to be made when necessary.

A rectangular grid coordinate system (similar to our 'flat Earth' grid) is associated with every map projection. Map projection coordinates are described in terms of Northing and Easting, being distances to the North and East of an origin. They are usually expressed in units of metres.

There are a large number of different types of map projections, each one representing a different way of distorting the surface of the ellipsoid into a plane. One of the most commonly used is the Transverse Mercator Projection.

Revisions of this Manual

This Manual was produced by FIG Commission 5. It is intended that it be revised and updated at regular intervals.

Any feedback suggestions on content should be forward to FIG Commission 5 Chair – see http://www.fig.net/commission5/

2 GEODESY AND GLOBAL REFERENCE FRAMES

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Surveyors increasingly use satellite positioning systems that provide position in terms of global reference frames. It is becoming increasingly important for surveyors to understand these reference frames and how they relate to local reference frames. This section gives an overview of the science of geodesy and use of global reference frames.

Modern Geodesy and the ITRS/ITRF

The primary mission of Modern Geodesy is the definition and maintenance of precise geometric and gravimetric reference frames and models, and the provision of high accuracy positioning techniques for users in order to connect to these frames.

The International Association of Geodesy (IAG) has established services for all the major satellite geodesy techniques: International Global Navigation Satellite System (GNSS) Service (IGS); International Laser Ranging Service (ILRS); International Very Long Baseline Interferometry Service (IVS); International Doppler Orbitography and Radio positioning Integrated by Satellite (DORIS) Service (IDS); the International Gravity Field Service (IGFS); and others. These services generate a wide range of products, including precise satellite orbits, ground station coordinates, Earth rotation and orientation values, gravity field quantities and atmospheric parameters, all of which are vital to the definition of the terrestrial and celestial reference systems. These reference systems are the foundation for all operational geodetic applications associated with mapping and charting, navigation, spatial data acquisition and management, as well as support for the geosciences.

The International Celestial Reference System (ICRS) forms the basis for describing celestial coordinates, and the International Terrestrial Reference System (ITRS) is the foundation for the definition of terrestrial coordinates to the highest possible accuracy. The definitions of these systems include the orientation and origin of their axes, scale, physical constants and models used in their realization, e.g., the size, shape and orientation of the reference ellipsoid that approximates the geoid and the Earth's gravity field model. The coordinate transformation between the ICRS and ITRS is described by a sequence of rotations that account for variations in the orientation of the Earth's rotation axis and its rotational speed.

While a reference system is a mathematical abstraction, its practical realization through geodetic observations is known as a reference frame. The conventional realization of the ITRS is the International Terrestrial Reference Frame (ITRF), which is a set of coordinates and linear velocities of well-defined fundamental ground stations. In the case of the ITRF these are the observatory stations of the IGS, ILRS, IVS, IDS ground networks, derived from space-geodetic observations collected at these points, and computed and disseminated by the International Earth Rotation and Reference Systems Service (IERS).

The solid surface of the Earth consists of a number of large tectonic plates (and many smaller ones whose boundaries are less well defined) that slide across the lithosphere, in the process colliding with other plates. The speed of the plates may be as high as a decimeter or more per year, though typically tectonic plate motion is of the order of a few centimeters per year relative to a fixed coordinate framework. That framework is realized by the fixed axes of the ITRF – defined in an inverse sense by tracking the orientation of the axes relative to the Earth's (moving) crust, and tracking of the location of the origin of the Cartesian system with respect to the (moving) geocenter, using geodetic techniques.

There have been several different realizations of the ITRF since 1989, each designated as ITRFyyyy where yyyy refers to the year of observation for the most recent data used in the computation of station coordinates and velocities. This may be different to the reference epoch, which is the date to which station coordinates and velocities are referenced.

Initially computed on an annual basis, since 1997 the new ITRF realizations have been released by the IERS at 3–5 year intervals, with the latest being ITRF2008, and a new ITRF2013 expected to be released in late 2014. Although each successive ITRF is more internally accurate than the previous one, the primary difference in the coordinates of the ground stations between different ITRFs reflects the motion of stations due to local crustal deformation and global plate tectonics between the reference epochs of the two frames.

ITRF Stations

A modern datum is defined by the 3D coordinates of a global, national or local network of fundamental stations at an instant in time. It may be the same epoch year as the ITRFyyyy reference frame, or any arbitrary epoch. Today it is comparatively easy to compute the coordinates of any ground station in a geocentric reference frame such as ITRF, at that instantaneous epoch of measurement using GNSS technology. See chapter 10.

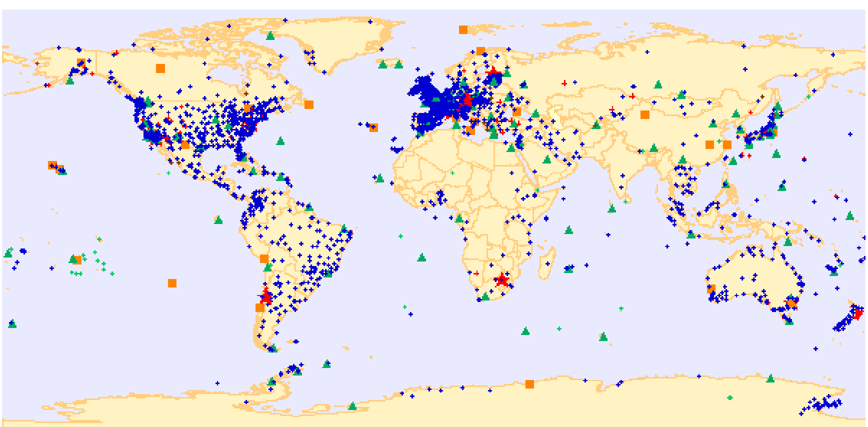

Figure 1: *Geodetic reference stations.*

Global Geodetic Observing System

In order to address the ever increasing performance requirements for global change monitoring, the IAG established in 2007 the Global Geodetic Observing System (GGOS). GGOS's goal is to coordinate all of the geodetic services and provide high-level products through a single portal. It will also fuel the next revolution in modern geodesy – the unified analysis of all geodetic data through common models – so as to drive an order of magnitude improvement in geodetic accuracy (reference frame stability, quality of geodetic products and models, etc.). In other words, GGOS will promote the development of tools and observing systems to ensure the ITRF can be defined and monitored to millimetre accuracy, with stability at the mm/yr level.

The mission of GGOS is: (1) to provide the observations needed to monitor, map and understand changes in the Earth's shape, rotation and mass distribution; (2) to improve the quality of the global reference frames so that they may provide the fundamental backbone for measuring and interpreting key global change processes; and (3) to support a variety of applications in geoscience and society for precise positioning, gravity field mapping and modeling.

The resultant improvement in reference frame accuracy and stability will not only benefit critical scientific studies such as measuring sea level rise, but also will provide a stronger framework for precise (centimeter-level) GNSS-enabled positioning in national, regional or global datums.

At a practical level the integration of the outputs of all the IAG services implies a coordinated upgrade of the ground station infrastructure (the stations in the IGS, ILRS, IVS,

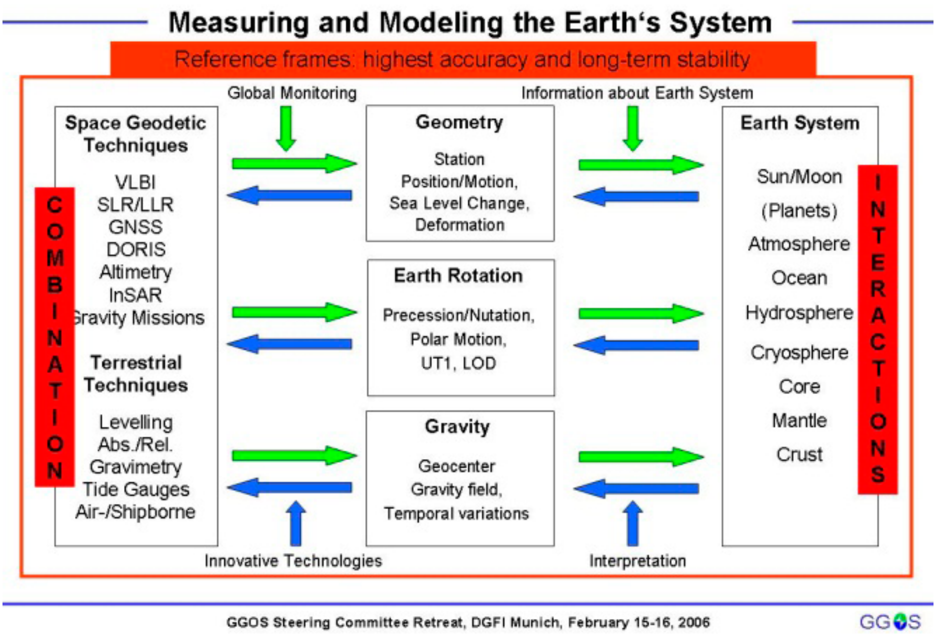

Figure 2: *Function of reference frame in measuring and modeling Earth systems.*

IDS networks); an increase in the number of co-located stations of the different space geodesy techniques; and steady improvement and continuity of a number of critical geodetic satellite missions (altimetric, GNSS, gravity mapping, etc). Under the GGOS initiative increased investment in geodetic infrastructure such as GNSS permanent reference stations is strongly encouraged.

Further Information

- International Association of Geodesy (IAG)
  www.iag-aig.org
- International GNSS Service (IGS)
  www.igs.org
- International Laser Ranging Service (ILRS)
  www.ilrs.gsfc.nasa.gov
- International VLBI Service (IVS)
  www.ivscc.gsfc.nasa.gov
- International DORIS Service
  www.ids-doris.org
- International Earth Rotation and Reference Systems Service (IERS)
  www.iers.org
- International Gravity Field Service (IGFS)
  www.igfs.net
- International Terrestrial Reference Frame (ITRF)
  itrf.ensg.ign.fr
- Global Geodetic Observing System (GGOS)
  http://www.ggos.org

3 GLOBAL TERRESTRIAL REFERENCE SYSTEMS AND FRAMES

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It is often necessary to transform between different global reference frames. This section provides a more in-depth overview for the surveyor who wishes to have knowledge of global terrestrial reference systems and frames and transformations between them.

Introduction

The International Earth Rotation and Reference Systems Service (IERS) was established in 1988 with a mission of providing:

- The International Celestial Reference System (ICRS) and its realization, the International Celestial Reference Frame (ICRF).
- The International Terrestrial Reference System (ITRS), and its realization, the International Terrestrial Reference Frame (ITRF).
- Earth orientation parameters (EOPs) to describe the time-varying relationship between the ICRF and ITRF.
- And standards, constants, and models for international consistency, as well as related geophysical data for interpretation.

The ICRF consists of equatorial coordinates of extragalactic radio sources observed with VLBI forming a nearly inertial frame while the ITRF consists of a set of three dimensional Cartesian coordinates and corresponding velocities which, in theory, strive to form an ideal Earth-centered, crust-fixed reference frame.

Relationships between Global Reference Frames

A Euclidean similarity transformation can relate two non-deformable Cartesian reference frames via seven parameters. The parameters are defined by three translation components $T = (T_1 \;\; T_2 \;\; T_3)^T$, three differential rotation angles (R_1, R_2, R_3) and one scale term, *D*. As an example, if a coordinate vector $X_1 = (x_1 \;\; y_1 \;\; z_1)^T$ is represented in one reference frame, the same coordinate can be represented in the second reference frame as $X_2 = (x_2 \;\; y_2 \;\; z_2)^T$ by using the following equation (linearized assuming relatively small transformation values):

$$X_2 = X_1 + T + DX_1 + RX_1 \tag{1}$$

with:

$$T = \begin{pmatrix} T_1 \\ T_2 \\ T_3 \end{pmatrix} \text{ and } R = \begin{pmatrix} 0 & R_3 & -R_2 \\ -R_3 & 0 & R_1 \\ R_2 & -R_1 & 0 \end{pmatrix} \tag{2}$$

The differential rotation matrix R is consistent with positive counterclockwise (anti-clockwise) rotation of axes. ** The IERS uses the opposite rotational sign convention.

In practice, the seven transformation parameters change with time, thus, differentiating Eq. (1) with respect to time it can be written:

$$\dot{X}_2 = \dot{X}_1 + \dot{T} + \dot{D}X_1 + D\dot{X}_1 + \dot{R}X_1 + R\dot{X}_1 \tag{3}$$

D and R are of 10^{-5} level and $\dot{X}$ is about 10 cm per year, the terms and $R\dot{X}_1$ are negligible, representing 0.1 mm over 100 years. Therefore, Eq. (3) can be reduced to:

$$\dot{X}_2 = \dot{X}_1 + \dot{T} + \dot{D}X_1 + \dot{R}X_1 \tag{4}$$

General Transformation between Coordinates X1 and X2

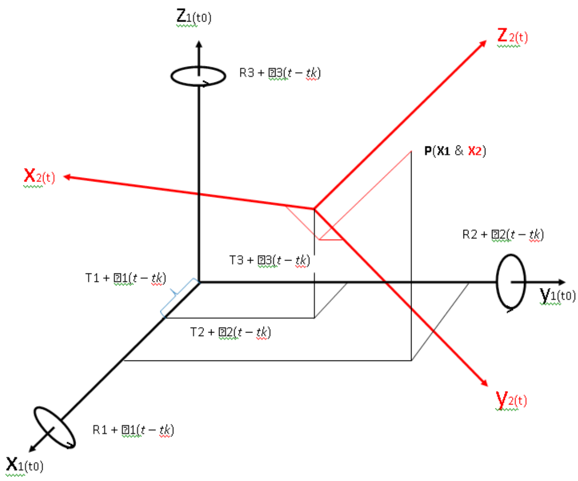

Figure 1: *Axes of reference frame.*

International Terrestrial Reference Frame (ITRF)

The ITRF is a global, geocentric 3D reference frame co-rotating with the Earth's crust in its diurnal motion in space. The most recent realization of the ITRF is the ITRF2008, with the reference epoch 2005.00 and full variance-covariance information. This frame is equivalent to GPS-determined IGS08 in terms of datum but positional offsets have been applied to the latter to compensate for a change in antenna calibrations made by the International GNSS Service (IGS) when IGS08 was adopted in April 2011. The ITRF2008 frame is specified as follows:

1. Origin: At the long-term average geocenter (center of mass of the total Earth system including its envelope of liquid and gaseous fluids). This point is realized through the dynamical motions of the satellites tracking by satellite laser ranging (SLR).
2. Z-axis: Originally directed toward the conventional definition of the North Pole, or more precisely, towards the conventional terrestrial pole (CTP) as defined by

the International Earth Rotation Service (IERS) but now realized by continuity (in offset and rate) of each ITRF realization with its predecessor.
3. X-axis: Passes through the point of zero longitude (approximately of the Greenwich meridian) as defined by the IERS (and also realized by continuity.)
4. Y-axis: Forms a right-handed coordinate system with the x- and z-axes (and also realized by continuity).

Geodetic Techniques Contributing to the ITRF

There are four space geodetic techniques that contribute to defining the ITRF, all on a global scale. They are:

1. Global Navigation Satellite Systems (GNSS) – consists of a constellation of at least 24 satellites in six orbital planes, which disseminates continuous worldwide navigation and timing information. The system provides very accurate, 3-D positional information to support the military as well as the civilian and commercial users.
2. Very Long Baseline Interferometry (VLBI) – consists of an array of very large radio telescopes that receive continuum microwave electromagnetic waves from extra-galactic quasars and pulsars to measure interferometric delay observations via direct cross-correlation of the signal streams.
3. Satellite Laser Ranging (SLR) – consists of a network of stations that measure the round trip time of flight of very short laser pulses between ground stations and Earth-orbiting satellites and the Moon with retro-reflectors. This technique is one of the most accurate in determining the position of satellites as well as determining temporal variations in the Earth's gravity field and the geocenter, and is also used for many Earth-monitoring missions such as ocean altimetry.
4. Doppler Orbitography and Radio positioning Integrated by Satellite (DORIS) – is a Doppler tracking system with a primary purpose of determining precise orbits and ground locations. Microwave signals are transmitted from a network of about 60 terrestrial beacons and observed by receivers on satellites orbiting the Earth. DORIS is routinely used to measure orbits of Earth monitoring satellites (with radial accuracies at the 1 cm level), such as for oceans topography.

World Geodetic System 1984 (WGS84)

The World Geodetic System of 1984 was designed by the National Geospatial Intelligence Agency (NGA) with the primary purpose of supporting the U.S. Department of Defense (DoD) and the armed forces. The reference frame is very similar to ITRF in specification. The latest realization was released on February 8, 2012 and is known as WGS 84 (G1674). Routine updates to WGS 84 are performed by NGA to align and conform to the latest realization of the International Terrestrial Reference Frame (ITRF).

Further Information

- An overview of GNSS reference frames is provided by the International Committee on Global Navigation Satellite Systems (ICG) - http://www.oosa.unvienna.org/oosa/en/SAP/gnss/icg/regrefsys.html

- International Earth Rotation and Reference Systems Service (IERS)
  www.iers.org
- The International Celestial Reference Frame (ICRF)
  www.iers.org/IERS/EN/DataProducts/ICRF/ICRF/icrf.html?__nnn=true

4 REGIONAL AND NATIONAL REFERENCE FRAMES

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Surveyors often make measurements in terms of regional or national reference frames. As our ability to make more precise measurements over longer distances continues to improve we need to be concerned with accommodating the effects of crustal deformation in our datums. This section provides information for the surveyor on the different types of reference frames and datums and how crustal deformation can be accommodated in them.

Global Reference Frames

A global reference frame is typically the primary basis for the definition of a coordinate system used in applied geodesy. Examples include the International Terrestrial Reference Frame (ITRF) and the World Geodetic System 1984 (WGS 84). These frames are geocentric in nature, having the geocenter (the center of mass of the Earth) as the origin and orthogonal axes aligned with pole, equator and Greenwich meridian according to IERS conventions. The ITRF is realized by the coordinates and site velocities of a network of global stations and forms the basis for modern regional and national reference frames or geodetic datums. The most recent realizations of ITRF have positional uncertainties of contributing stations in the order of millimeters. ITRF station velocities are described with respect to a no-net-rotation (NNR) condition where the angular momenta of all of the global tectonic plates sum to zero.

Regional Reference Frames

Regional reference frames are denser networks of geodetic stations covering continental areas. Examples include the European Terrestrial Reference Frame (EUREF), North American Datum 1983 (NAD83), African Reference Frame (AFREF), Sistema de Referencia Geocéntrico para las América (SIRGAS) and the Asia-Pacific Reference Frame (APREF). As with ITRF, regional reference frames are defined by the coordinates and site velocities of contributing stations. The key difference with some regional reference frames (e.g. EUREF and NAD83) and ITRF is that the site velocities may be with respect to the dominant tectonic plate encompassed by the frame and not a NNR condition. This approach minimizes site velocities. Regional frames not constrained by the motion of a single tectonic plate are closely aligned with ITRF.

National Reference Frames

Modern national reference frames are typically a static realization of ITRF or a regional reference frame. In most countries the coordinates of a national reference frame (or geodetic datum) form the basis for all surveying, positioning and mapping within national borders. Because surveying/ GIS software and spatial data are not generally designed to deal with continuously changing coordinates, the epoch for national datums is fixed and the coordinates are considered to be invariant with time.

Types of Geodetic Datums

Static geodetic datums aligned with a fixed epoch (or reference epoch) of an ITRF realization currently support most spatial users; however there are some practical limitations with a static datum if GNSS positioning techniques are used. GNSS point positioning uses orbit models defined in ITRF or WGS84 reference frames. As a consequence, the precise location within these frames will change as a function of time due to tectonic processes and other deformation sources such as subsidence, soil creep and post glacial uplift. Centimeter level precise positioning is rapidly becoming more widely accessible and inexpensive. Unless precise GNSS positions are localized (e.g. via a local CORS or site transformation at a geodetic reference mark), users will notice positions of 'fixed' (in a local reference frame) objects changing every few months. Rigid tectonic plates rotate slowly over the Earth's mantle however the rotation is rapid enough to introduce errors in static GNSS baseline processing and RTK over long baselines if the rotation rate is high and there is large interval between the measurement and reference epochs.

In many locations near tectonic plate boundaries such as the Western states of the USA, Chile, Japan, Indonesia, China, Papua New Guinea, New Zealand, Greece, Pakistan and North Africa, the assumption of a static datum at a national level is inappropriate. In these locations the magnitude of plate boundary deformation can be as much as several cm/year between any two geodetic stations in the national network, with meters of deformation possible after large earthquakes.

To enable precise GNSS positions to be used within the context of a static geodetic datum, these complex deformations are required to be modelled in some way. A **semi-kinematic datum** (or semi-dynamic datum) is one where a deformation model forms an integral part of the datum definition. GNSS point positions determinations and geodetic data analysis is undertaken in the kinematic ITRF reference frame, using the latest realization of ITRF. Resulting coordinates are then propagated back to the fixed reference epoch of the semi-kinematic datum, so that spatial data can be integrated seamlessly over long periods of time. The utility of the semi-kinematic datum approach is that precision data analysis is not degraded as a result of un-modelled deformation, however to the end user, the geodetic datum appears to be static at a fixed reference epoch.

A **fully kinematic datum** overcomes a lot of the limitations of unmodelled deformation in positioning, however the major limitation (currently) is that it is very difficult to integrate spatial data acquired over a longer period of time, unless a precise deformation model is embedded in the data or somehow explicitly referenced. Inclusion of deformation models in GIS and spatial data infrastructures is an area of current research, although implementation is still some time away. Until then, a more practical and logical approach is to apply a deformation model after GNSS data capture, so that the resulting coordinates can be used in 'static' spatial data frameworks and GIS.

Deformation Models

There are currently several approaches to using deformation models in practical geodetic applications. For regions located on rigid tectonic plates, a rigid plate rotation model (3 parameters) defining the Euler pole of rotation can be used. This approach is currently being adopted by Trimble in their RTX service. Another approach is to use

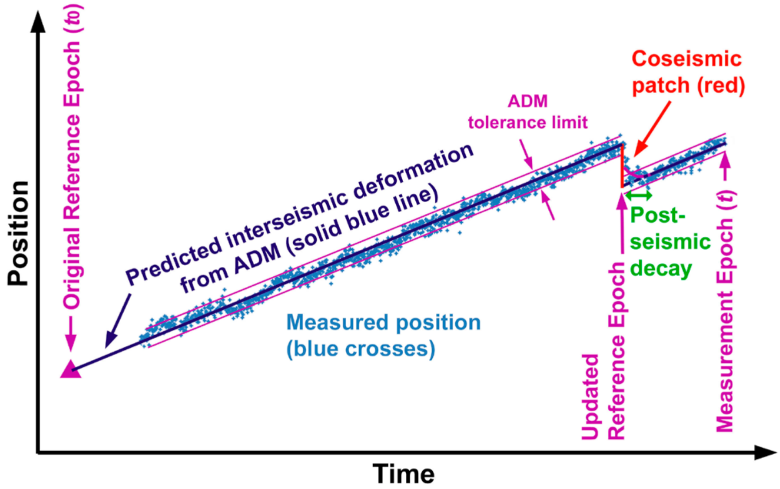

Figure 1: *Observed and modeled deformation through time.*

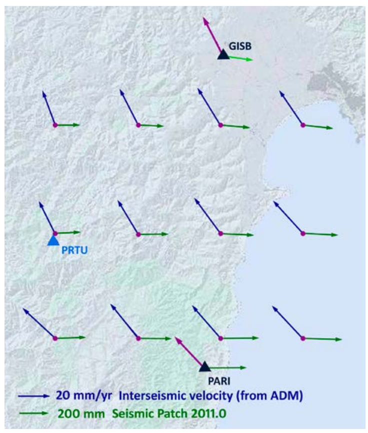

Figure 2: *Interseismic velocities and seismic patch deformation.*

a 14 parameter transformation (7 parameter conformal transformation plus their respective rates of change). This is the current approach used in Australia to transform ITRF coordinates at the measurement epoch to GDA94 which is a static datum fixed at epoch 1994.0.

Rigid plate transformations do not work satisfactorily in areas where internal deformation is occurring or is complex in nature (Fig. 1). A gridded deformation model approach is currently used in New Zealand to enable transformation from ITRF at the measurement epoch to NZGD2000 (epoch 2000.0). Gridded deformation models can have variable resolution to suit the complexity of deformation and the precision requirements of users. Such a deformation model is computed primarily from dense geodetic networks with repeat observations made over a long enough period of time to estimate the ITRF site velocities for stations in the network. These site velocities can then be analyzed and inverted to estimate Euler poles of rigid crustal blocks, slip distribution and then subsequently site velocities over a regular grid. A typical deformation model is described by a grid of latitudes and longitudes with associated site velocities in East (longitude), North (latitude) and Up (elevation) components.

Episodic deformation events such as earthquakes require supplementary deformation modeling. These events are often significant at localized scales, and may warrant a permanent change in the coordinates at a post-event epoch to reflect reality on the ground. An episodic deformation model (or patch model) can account for these localized deformation events and use of the patch model enables GNSS technology to be used in a pre-earthquake spatial infrastructure by applying the patch model in reverse mode. The patch model is defined in much the same way as the site velocity model but with deformation magnitudes fixed to cover the period of the earthquake, after-shocks and post-seismic deformation (Fig. 2). Again, the source data for the patch model are typically repeat GNSS measurements at a dense network of passive geodetic stations. However, controlled InSAR, LiDar, high resolution imagery and geophysical models can also be used to define a grid of surface deformations associated with a deformation event.

5 HEIGHT SYSTEMS

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Traditionally the surveyor is interested in determining heights in terms of sea level. This is of interest for reasons related to water flow, for example, how high is a building above a flooding river? However satellite positioning systems determine heights relative to the ellipsoid and these must be converted to the more useful sea level heights for many applications. This section describes the various heights systems and how heights can be transformed between these systems.

The Relationship between Gravity and the World Height System (WHS)

Traditional leveling, height systems, and vertical datums relate to positioning in the Earth's gravity field. Moving away from the Earth's center (geocenter) produces a lower potential of gravity (geopotential), which is directly related to the height change. Geopotential is measured in units of m^2/s^2, and is usually divided by some value of gravity to arrive at a height e.g.

$$m^2/s^2 \div m/s^2 = m$$

Normal gravity from the ellipsoid (γ) gives normal heights. The mean gravity value from the surface down to the geoid along the plumb line gives orthometric heights. A constant value of gravity (usually γ at latitude 45) gives dynamic heights. These heights are all different, but they all use the same geopotential numbers. So unification of height systems could be based on a Global or Earth Gravity Model (GGM/EGM).

One such model currently available is the Earth Gravity Model of 2008 (EGM2008) produced by the US National Geospatial-Intelligence Agency (NGA). Such a model can be used to generate various aspects of the Earth's gravity field including geoid heights and gravity anomalies.

How GNSS Measurements Are Linked to Local Orthometric Heights

The primary link is through the geoid height model. As noted previously, a geoid height is measured from a specific ellipsoid surface to a specific geoid surface or vertical datum. Hence geoid height models are ellipsoid heights and intrinsically tied to a specific geometric reference frame. Ellipsoid heights (h) determined using GNSS techniques in a specified reference frame can be transferred by means of the associated geoid heights (N) to estimate orthometric heights (H) according to the below formula:

$$H = h - N$$

The order of operations is then:

– Obtain your position using GNSS in the required reference frame

– Interpolate the geoid height model to the determined latitude & longitude

- Subtract the interpolated geoid height from the observed ellipsoid height
- This nets the desired orthometric height

The Relationships Among the Various Aspects of the Earth's Gravity Field

The average Earth model is best described by an ellipsoid of rotation. It has a certain mass, spin rate, and shape that matches the real Earth to 99.999%. However, the missing bit is equal to +/– 100 meters, which is significant at human scales. The difference arises from the lack of uniformity in the way the Earth's mass is distributed. Mass is the key. Mass variations are directly related to all aspects of the earth's gravity field. If the earth were uniform in distribution of mass, then the ellipsoid model might suffice. However, the Earth's mass varies greatly and is of interest.

Thinner submerged oceanic rock lies next to continental rock, which is ten times thicker. This is a long wavelength gravity signal, because it describes features at continental and oceanic scales. The Earth's topography provides a lot of short wavelength gravity signal, because the density contrast between rock and air is greater by far than the contrast between different types of rock. The gravitational effect of an entire mountain range is long wavelength, but the effect of a single mountain is short wavelength.

The cumulative effect of all the Earth's masses are expressed in different but related functions of the gravity field (called functionals). Gravity and geopotential are two functionals already covered, but values associated with these full signal functionals are quite large. Most gravity field applications remove some type of ellipsoid reference model value. The Geodetic Reference System of 1980 (GRS80) and World Geodetic System of 1984 (WGS84) are two such models.

The difference between actual gravity (g) and Normal gravity from the ellipsoid (γ) is a gravity anomaly (Δg). The difference between the real world geopotential (W) and ellipsoid potential (U) is the disturbing potential (T). Geoid heights (N) represent the difference between heights above the ellipsoid (h) and heights above the geoid or vertical datum (H).

$$\Delta g = g - \gamma$$

$$T = W - U$$

$$N = h - H$$

In turn, both geoid heights and gravity anomalies can be related to the disturbing potential:

$$\Delta g = -(\partial T)/(\partial r) - (2\,T)/r \quad N = T/\gamma$$

Both gravity anomalies and geoid heights are expressed in terms of geopotential and can be related through a complex formula called the Stokes equation. This is useful because gravity observations are easily made and manipulated to determine a geoid for use as a vertical datum.

How Geoid Models Are Determined and Used in Vertical Datum

Models like EGM2008 were made from many sources. Orbiting satellites such as GRACE and GOCE provided long wavelength signal. Surface gravity and topography models provided short wavelength signal. Combining them resulted in a sub-meter accurate model for 10 km and larger features. For improved accuracy, regional models are generally required.

Regional models are developed using an EGM as a reference model. They are enhanced by shorter wavelength signal that had to be omitted from the global model to keep the solution of the EGM feasible. Often times, the same surface gravity observations are used but more of the signal is retained in the final product.

Regional models reflect satellite data at the longest wavelengths (>200 km scales), which makes them consistent with any World Height System (WHS). Then surface and airborne gravity data as well as terrain models fill in the shortest wavelengths down to the resolution of the regional model (2–5 km scales). With the added information, regional geoid models can be sub-decimeter to sub-centimeter level accurate. This model can then be combined with GNSS observations to yield orthometric heights.

Further Information

The following web pages provide more help –

- EGM2008
  http://earth-info.nga.mil/GandG/wgs84/gravitymod/egm2008
- GRACE
  http://www.csr.utexas.edu/grace/
- GOCE
  http:/earth.esa.int/GOCE
- GRS-80
  http://www.gfy.ku.dk/~iag/HB2000/part4/grs80_corr.htm
- International Geoid Service
  http://www.iges.polimi.it/

6 TRANSFORMING BETWEEN DATUMS

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The surveyor is often required to transform data between different datums. This might be transforming newly derived coordinates in terms of an official datum to an older legacy datum or visa versa. This section provides information on the commonly used transformation methods and some of the more specific cases used for transforming between datums.

Introduction

Coordinates can be converted from one datum to another if the relationship between the two is known. The relationship is described by two components, being:

- a set of formulae which describe the mathematics of the transformation process,
- a set of parameters, referred to as transformation parameters that are used in the formulae.

Often more than one transformation may be defined to convert between datums. For example there may be simple low accuracy transformations, and more complex transformations to higher accuracy.

The transformation parameters are derived by analysing survey control stations which have coordinates on both datums. The minimum number of stations required for this process depends on the transformation method being proposed. However, there is no maximum limit, and in general, as many stations as possible will be used.

The accuracy of a transformation can be assessed by comparing transformed coordinates at points which have observed coordinates defined in both datums.

Consider the typical situation where we have:

- Coordinate Set 1 which has been computed from Network 1 on Datum 1,
- Coordinate Set 2 which has been computed from Network 2 on Datum 2.

If the coordinates from Set 1 are converted to Datum 2, they will generally not exactly match the coordinates of Set 2. This is because the coordinate sets are derived from different measurement sets or network geometries (for example, Network 1 may have been measured by triangulation and Network 2 by GPS). One network will appear offset and distorted relative to the other.

The differences between the transformed Datum 1 coordinates and the network Datum 2 coordinates are known as residuals. Their magnitude provides an indication of the quality of the network held in the two datums, as well as an indication of the accuracy of the transformation between those datums.

Transformation parameters are commonly generated by government mapping organisations and are freely available to users. They are also commonly embedded in transformation software, such as Geographic Information Systems (GIS).

Similarity Transformations

Modern three dimensional datums are commonly related using similarity transformations, which allow for a difference in location, rotation, and scale between the two datums, but do not allow any distortion in them. These include three, seven, and 14 parameter transformations.

The seven parameter transformation expresses the relationship between the two datums in terms of a translation, rotation, and scale factor. The translation and rotation are defined for each of the 3 coordinate axes, making a total of 7 parameters.

The following formulae are used to convert cartesian coordinates from Datum 1 to Datum 2 values using a seven parameter transformation (Fig. 1):

$$\begin{bmatrix} X_2 \\ Y_2 \\ Z_2 \end{bmatrix} = \begin{bmatrix} T_X \\ T_Y \\ T_Z \end{bmatrix} + \left(1+\Delta_S\right) \begin{bmatrix} 1 & -R_Z & +R_Y \\ +R_Z & 1 & -R_X \\ -R_Y & +R_X & 1 \end{bmatrix} \begin{bmatrix} X_1 \\ Y_1 \\ Z_1 \end{bmatrix}$$

where

X_1, Y_1, Z_1	are the Cartesian coordinates of Datum 1
X_2, Y_2, Z_2	are the Cartesian coordinates of Datum 2
T_X, T_Y, T_Z	is the difference between the centres of the two ellipsoids
R_X, R_Y, R_Z	are the rotations around the three coordinate axes in radians,
Δ_S	is the scale difference between the coordinate systems as a ratio.

Typically the rotations are published in units of arc seconds or milli-arc seconds (mas), and the scale as parts per million (ppm) or parts per billion (ppb).

Note that these formulae, commonly used in geodesy, are simplified on the assumption that the rotations are small (less than a few arc seconds).

Note also that there is ambiguity in the sign convention for rotations. Rotations supplied by some agencies may need to be negated to be used in these formulae.

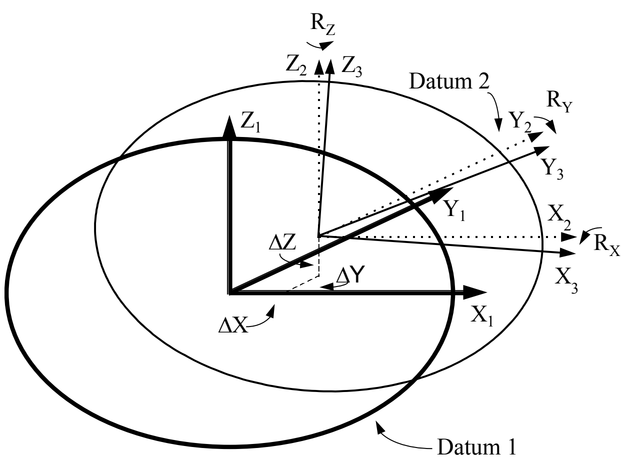

Figure 1: *Transformation between datum 1 and datum 2.*

The 14 parameter transformation is a variation of the seven parameter transformation in which each parameter is assigned both a value at a reference epoch and a rate of change. For example the rotation about the X axis will be represented by its value R_{X0} at the reference epoch t_0 and its rate of change R'_{X0}. The rotation R_X at time t is calculated as

$$R_X = R_{X0} + (t - t_0)\, R'_{X0}$$

The 14 parameter transformation is often used to express the relationship between modern high accuracy datums, such as the International Terrestrial Frame (ITRF) realisations – see http://itrf.ensg.ign.fr/doc_ITRF/Transfo-ITRF2008_ITRFs.txt.

The three parameter transformation is a special case of the seven parameter transformation in which the rotations and scale difference are 0 – the datums differ only in location.

Other Transformation Methods

The three, seven, and 14 parameter transformations apply between three dimensional datums, typically established by GNSS surveys. That is they assume that the datum defines ellipsoidal heights as well as latitude and longitude. Also they do not account for possible distortion in the datums (other than a difference in scale).

Older datums are often two dimensional – they define latitude and longitude but not ellipsoidal height. They may be distorted due to the limitations of the survey techniques with which they were established, and due to deformation that has occurred since then.

These datums may still have three or seven parameter similarity transformations defined relating them to their modern counterparts. While these cannot account for the distortion, they are well supported by survey software, and may be sufficiently accurate for many users. When using such transformations the height ordinate may be ignored. A more complex definition is needed for accurate transformations between such datums.

One commonly used approach is based on a transformation grid. This is a grid of points defined at latitudes and longitudes in Datum1. At each grid point the corresponding latitude and longitude at Datum2 is defined. The transformation at other points is defined by interpolating across the grid cell in which the point lies.

Grid transformations are often published using the NTv2 format. This is used in many jurisdictions and supported by a wide range of GIS software – for more details see https://www.nrcan.gc.ca/earth-sciences/geomatics/geodetic-reference-systems/tools-applications/9074.

Another example of a transformation method used in Sweden is to represent the relationship between a local grid datum and a national geodetic datum using a best fitting Transverse Mercator projection – see http://www.lantmateriet.se/Global/Kartor%20och%20geografisk%20information/GPS%20och%20m%c3%a4tning/Geodesi/Rapporter_publikationer/Rapporter/Rapport_Reit_eng.pdf

7 TRANSFORMING BETWEEN DATUMS IN NON-STATIC REFERENCE FRAMES

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Surveyors are increasingly working in non-static reference frames, reference frames that account for the effects of crustal movements. This section details the specific case for transforming between these non static reference frames.

Introduction

Non- static reference frames may be described using a number of terms, such as kinematic or semi-kinematic (or dynamic, semi-dynamic). What they all have in common is a means (and requirement) to account for the fact that the Earth is always moving.

Use Cases

Often, government organizations or software vendors will make tools available to facilitate transformations. However, there are situations where the surveyor needs to understand and be able to carry out these transformations. For example:

1. New software or equipment is being tested
2. Official models used in transformations are not sufficiently accurate for the project area
3. Software or online transformation tools are unavailable

General Example

Getting precise coordinates in the latest ITRF realization has been greatly simplified through the provision of online GNSS processing services. Many of these provide absolute position, using the Precise Point Positioning (PPP) technique. However, most jurisdictions or positioning applications require coordinates in terms of a local reference frame. While we could make relative connections to control provided by the national geodetic agency, this is not always the most efficient method. A transformation can be used to convert the ITRF coordinates to the local reference frame.

Example

Submitting GNSS data to an online PPP processing service, enables ITRF2008 (Epoch 2012.16) coordinates for seven existing and three new stations in New Zealand to be obtained. In the area of our survey, the official reference frame is ITRF96 (Epoch 2000.0). Each of the seven existing stations has published values for ITRF96 (Epoch 2000.0). There is a gridded velocity model available for our area of interest (Fig. 1).

Note that throughout this example, coordinates and velocities are in terms of the geocentric reference frame.

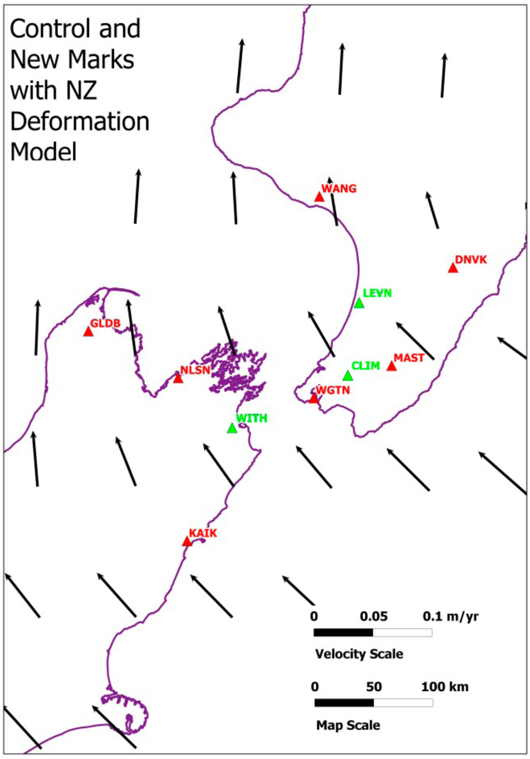

Figure 1: *Existing stations (red triangles), new stations (green triangles) and gridded velocities.*

Step 1 Calculate ITRF96 Epoch 2012.16 Coordinates

In this step we use the official velocity model and published ITRF96 (Epoch 2000.0) coordinates to calculate ITRF96 coordinates at the same epoch for which we have observed ITRF2008 coordinates (Epoch 2012.16), using Equation (1):

$$\mathbf{x}_{\text{ITRF96 Epoch 2012.16}} = \mathbf{x}_{\text{ITRF96 Epoch 2000.0}} + 12.16\mathbf{v}_{XYZ} \tag{1}$$

Using Equation (1) for station GLDB:

$$\begin{bmatrix} x \\ y \\ z \end{bmatrix}_{2012.16} = \underbrace{\begin{bmatrix} -4792405.831 \\ 628416.781 \\ -4148068.669 \end{bmatrix}}_{\text{Published ITRF96 Epoch 2000.0 coordinates}} + 12.16 \underbrace{\begin{bmatrix} -0.0285 \\ 0.0045 \\ 0.0333 \end{bmatrix}}_{\text{Interpolated from published velocity model}} = \begin{bmatrix} -4792406.177 \\ 628416.835 \\ -4148068.263 \end{bmatrix}$$

Step 2: Calculate transformation parameters between ITRF96 and ITRF2008 at Epoch 2012.16

Here we use the well-known least squares method to estimate transformation parameters. In this case we choose to estimate only three parameters, due to the limited extents of the survey area when compared to the Earth as a whole.

- We use least squares to obtain the best solution, as we have more observations than parameters
- Functional model: **At** = **b**, where **A** is the design matrix, **b** = Calculated (ITRF96) minus observed (ITRF2008) and **t** is the matrix of unknown transformation parameters
- Stochastic model: **W** = **I**, in this case we choose to weight all coordinates equally
- So we have the standard least squares equations:

  $\mathbf{t} = (\mathbf{A}^T\mathbf{A})^{-1}\mathbf{A}^T\mathbf{b}$ (2)

  $\mathrm{Cov}(\mathbf{t}) = s_0^{\,2}(\mathbf{A}^T\mathbf{A})^{-1}$ (3)

  Aposteriori Standard Error of Unit Weight,
  $s_0^{\,2} = (\mathbf{A}^T\mathbf{t}-\mathbf{b})^T(\mathbf{A}^T\mathbf{t}-\mathbf{b})/(\text{degrees of freedom})$ (4)

- This is a linear problem, so no need to iterate

$$\begin{array}{ll}
GLDB & x \\ & y \\ & z \\
NLSN & x \\ & y \\ & z \\
KAIK & x \\ & y \\ & z \\
WGTN & x \\ & y \\ & z \\
MAST & x \\ & y \\ & z \\
DNVK & x \\ & y \\ & z \\
WANG & x \\ & y \\ & z
\end{array}
\quad A = \begin{bmatrix}
1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \\
1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \\
1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \\
1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \\
1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \\
1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \\
1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1
\end{bmatrix}
\quad b = \begin{bmatrix}
-4792406.177 \\ 628416.835 \\ -4148068.263 \\
-4775888.435 \\ 549740.211 \\ -4177981.109 \\
-4685479.521 \\ 531055.197 \\ -4280819.034 \\
-4777269.652 \\ 434270.387 \\ -4189484.267 \\
-4801933.943 \\ 370789.222 \\ -4167752.305 \\
-4860760.939 \\ 325692.752 \\ -4103646.312 \\
-4888073.52 \\ 443004.771 \\ -4060015.325
\end{bmatrix}
-
\begin{bmatrix}
-4792406.117 \\ 628416.851 \\ -4148068.23 \\
-4775888.398 \\ 549740.200 \\ -4177981.061 \\
-4685479.471 \\ 531055.245 \\ -4280819.009 \\
-4777269.602 \\ 434270.406 \\ -4189484.221 \\
-4801933.888 \\ 370789.24 \\ -4167752.257 \\
-4860760.892 \\ 325692.771 \\ -4103646.255 \\
-4888073.493 \\ 443004.775 \\ -4060015.31
\end{bmatrix}
=
\begin{bmatrix}
-0.06 \\ -0.016 \\ -0.033 \\
-0.037 \\ 0.011 \\ -0.048 \\
-0.05 \\ -0.048 \\ -0.025 \\
-0.05 \\ -0.019 \\ -0.046 \\
-0.055 \\ -0.018 \\ -0.048 \\
-0.047 \\ -0.019 \\ -0.057 \\
-0.027 \\ -0.004 \\ -0.015
\end{bmatrix}$$

Calculated ITRF96 Epoch 2012.16 coordinates

Observed ITRF2008 Epoch 2012.16 coordinates

The calculated transformation parameters using Equations (2), (3) and (4) are:

- SEUW = 0.015 m
- t_x = -0.046 ± 0.006 m
- t_y = -0.016 ± 0.006 m
- t_z = -0.039 ± 0.006 m
- Note: In this case least squares simply gives us the average of the coordinate differences, so we could have avoided the matrix algebra, but would not get the precision information so easily

Step 3: *Calculate survey epoch coordinate for new stations*

We now apply the transformation parameters to obtain an ITRF96 (Epoch 2012.16) coordinate for each new station:

$$\mathbf{x}_{\text{ITRF96 Epoch 2012.16}} = \mathbf{x}_{\text{ITRF2008 Epoch 2012.16}} + \mathbf{t} \tag{5}$$

Using Equation (5) for station CLIM:

$$\begin{bmatrix} x \\ y \\ z \end{bmatrix}_{ITRF96\ 2012.16} = \begin{bmatrix} -4793404.120 \\ 407108.010 \\ -4175081.520 \end{bmatrix} + \begin{bmatrix} -0.046 \\ -0.016 \\ -0.039 \end{bmatrix} = \begin{bmatrix} -4793404.167 \\ 407107.994 \\ -4175081.559 \end{bmatrix}$$

Step 4: *Calculate reference epoch coordinate for new stations*

$$\mathbf{x}_{\text{ITRF96 Epoch 2000}} = \mathbf{x}_{\text{NZGD2000 Epoch 2012.16}} - 12.16\mathbf{v}_{\text{xyz}} \tag{6}$$

Using Equation (6) for station CLIM:

$$\begin{bmatrix} x \\ y \\ z \end{bmatrix}_{ITRF96\,Epoch\,2000.0} = \begin{bmatrix} -4793404.167 \\ 407107.994 \\ -4175081.559 \end{bmatrix} - 12.16 \begin{bmatrix} -0.0196 \\ 0.0277 \\ 0.0250 \end{bmatrix} = \begin{bmatrix} -4793403.928 \\ 407107.657 \\ -4175081.864 \end{bmatrix}$$

8 REFERENCE FRAME PARAMETER ESTIMATION VIA THE TECHNIQUE OF LEAST SQUARES

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When establishing or propagating a reference frame from a set of measurements the surveyor is required to estimate station coordinates and their precisions, to test the quality of the measurements. These tasks are most commonly carried out using the method of Least Squares. This section focuses on the propagation of an international or regional reference frame onto national or local stations.

Introduction

Fundamental to the task of realising geodetic reference frames, whether they be local, national, regional or global, is the estimation of parameters for fundamental ground stations from repeated sets of measurements observed over those stations. The parameters of interest can include, for example, reference frame origin, orientation and scale, station coordinates, linear velocities, geoid-ellipsoid separations, datum distortions and deformation coefficients. The measurements are predominantly in the form of continuous measurement streams obtained from Global Navigation Satellite System (GNSS) Continuously Operating Reference Station (CORS) sites.

Whilst every geodesist/surveyor involved in the measurement process will always endeavour to obtain the most accurate and precise measurements, the true value of the parameters being estimated can never be derived with absolute certainty due to the inescapable influence of error on the measurement process. This holds true for even the most sophisticated and precise geodetic measuring techniques.

In order to achieve the most reliable estimate of station parameters and their uncertainties, whilst at the same time affording the geodesist a means for identifying and eliminating unacceptable measurement error, a rigorous and reliable estimation technique should be used. For the purpose of reference frame realisation, the technique of least squares is ideally suited to the task of estimating reference frame parameters.

Reference Frame Definition and Propagation

In the context of reference frame geodesy, an adjustment will usually be undertaken to satisfy one of two objectives – to *define* (or establish) a reference frame or to *propagate* a reference frame onto other stations. For the former, which is intended to establish the primary parameters of the reference frame (as in a particular realisation of ITRF), plus the positions, velocities and uncertainties of the fundamental stations, a large set of measurements from a variety of space and terrestrial geodetic measurement techniques will be included in the one solution. The realisation of ITRF is the responsibility of the International Earth Rotation and Reference Systems Service (IERS).

When propagating a reference frame onto other marks connected to the fundamental stations, the positions and uncertainties of the fundamental stations will be introduced into the solution as measured quantities. This is performed so that the location, orientation and scale of the reference frame will constrain the solution and thereby yield positions and uncertainties aligned to that frame. This is the procedure normally followed when establishing regional, national and local reference frames.

The Mathematical Model – the Vital Link between Geodetic Measurements and Reference Frame Parameters

The first step in estimating parameters using the least squares technique is to identify the mathematical model that appropriately relates measurements to the parameters of interest. In matrix form, the relationship between a set of measurements $\mathbf{m}$ and unknown parameters $\mathbf{x}$ can be written as:

$$m = Ax$$

where *A* is the matrix of coefficients describing the linear mathematical relationship between the two quantities. In classical geodesy, a mathematical model is comprised of a *functional model* and a *stochastic model*.

A **functional model** expresses the deterministic relationship between a measurement and the unknown parameters. In general terms, a measurement m can be expressed as:

$$m = f(x_1, x_2, \ldots, x_n)$$

where $x_1, x_2, \ldots, x_n$ are the parameters to be estimated and *f* is the function that relates the measurement to the unknown parameters. The functions used in geodesy vary considerably in purpose and complexity, ranging from unity to complex mathematical expressions. Measurement functions may also involve one or more unknown parameters and can contain several coefficients.

A **stochastic model** represents the statistical properties of the measurements and is used to determine the amount of contribution each measurement should have on the solution. The stochastic model may also include additional constraints and estimates of uncertainty to be imposed on the solution. Such estimates may be used to account for known but un-modelled systematic errors in the measurement process. In other cases, the constraints may be in the form of station position uncertainties derived from former solutions.

The stochastic model is represented by a variance matrix $\mathbf{V}$. The diagonal elements in the variance matrix are expressed in terms of standard deviations squared (i.e. variance). The off-diagonal terms represent statistical dependence (or correlation) between the respective variances. The formulation of GNSS variance matrices is usually handled by GNSS analysis software, although variances for conventional terrestrial measurements may require human intervention. Rigorous estimation of this matrix is of utmost importance to achieving the reliable estimates of station parameters and their uncertainties.

Whilst it is possible to estimate parameters from the functional model alone, the functional model and the stochastic model must be considered together if a rigorous and reliable estimate of the parameters is required. Since the parameters are a direct function of the measurements, it follows that the quality of the estimated parameters is a direct function of the quality of the measurements. In addition, the stochastic model provides the fundamental basis upon which measurements and estimated results are tested and assessed for measurement outliers and survey reliability. For the purposes outlined above, a *reliable* stochastic model is the cornerstone to *reliable* least squares estimation and testing.

Estimation and Adjustment

In a well-designed solution, the number of measurements (n) will exceed the number of unknown parameters (u) and as such, redundancy will exist. That is, the degrees of freedom ($r = n - u$) will be greater than zero. In any solution where there is redundancy, inconsistencies in the functional model will inevitably arise due to the presence of error in the measurements. Consequently, a number of possible values will exist for the parameters.

The purpose of least squares therefore, is to arrive at unique values of the parameters by estimating corrections (or adjustments) to each measurement such that the functional and stochastic models are satisfied. In matrix form, the least squares solution attempts to solve the most probable parameter values $\hat{\mathbf{x}}$ by satisfying the following:

$$\mathbf{m} + \hat{\mathbf{v}} = \mathbf{A}\hat{\mathbf{x}}$$

Here, $\hat{\mathbf{v}}$ denotes the corrections to the measurements. The least squares algorithm selects values for $\hat{\mathbf{v}}$ such that the unknown parameters $\hat{\mathbf{x}}$ will have maximum probability. This is satisfied by the following condition:

$$w = \mathbf{v}^T \mathbf{V}_{\mathbf{m}}^{-1} \mathbf{v} \rightarrow minimum$$

This quantity represents the weighted sum of the squares of the corrections to the original measurements, which is where least squares derives its name. By minimising w, the algorithm for the solution of unknown parameters $\hat{\mathbf{x}}$ from a set of measurements $\mathbf{m}$ and measurement variances $\mathbf{V}_{\mathbf{m}}$ can be written as:

$$\hat{\mathbf{x}} = \left(\mathbf{A}^T \mathbf{V}_{\mathbf{m}}^{-1} \mathbf{A}\right)^{-1} \mathbf{A}^T \mathbf{V}_{\mathbf{m}}^{-1} \mathbf{m}$$
$$\mathbf{V}_{\hat{\mathbf{x}}} = \left(\mathbf{A}^T \mathbf{V}_{\mathbf{m}}^{-1} \mathbf{A}\right)^{-1}$$

where $\mathbf{A}$ is a linearised design matrix representing the functional model, and $\mathbf{V}_{\hat{\mathbf{x}}}$ is the rigorous variance matrix for the estimated parameters.

The information contained in $\mathbf{V}_{\hat{\mathbf{x}}}$ is essential to understanding the spatial behaviour of uncertainty in the reference frame. It also provides a rigorous basis for the propagation of uncertainty into all other forms of spatial information aligned with a particular frame, and the means for making various inferences about the absolute and relative quality of geographic features.

References

Cooper, M. (1974). *Control Surveys in Civil Engineering*. Collins Professional and Technical Books, London.

Mikhail, E. (1976). *Observations and Least Squares*. IEP, Dun-Donnelley, New York.

9 TESTING MEASUREMENTS AND LEAST SQUARES PARAMETER ESTIMATES

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The section on Reference Frame Parameter Estimation via the technique of Least Squares gave a general overview of the topic. This section briefly reviews some basic concepts and techniques for the testing of geodetic measurements and least squares parameter estimates, and for estimating network reliability.

Introduction

The primary purpose of least squares estimation is to solve for a set of unknown parameters from a set of measurements. A secondary, although equally important purpose of least squares is to provide a basis for validating the *quality* and *reliability* of the system. In the context of reference frame geodesy, least squares validation involves testing (1) the quality of measurements and their *a-priori* precisions, and (2) the quality of the least squares solution as a whole. An additional validation task that may be undertaken is reliability testing, in which the measurements and the geodetic network are tested for *internal* and *external* reliability.

The reasons for testing are well known. Firstly, the assumed precision of a single or set of measurements may not be a realistic indication of the true measurement quality. Secondly, continuous streams of GNSS CORS measurements and repeated campaign measurements may indicate that one or more stations have moved and as a consequence, older measurements no longer represent the real world. In this instance, inconsistencies in the solution will arise and so a means for detecting measurement outliers will become necessary. Thirdly, it is possible that erroneous or sub–standard measurements may have been captured. Failure to detect and correct for the influence of any one of these factors can lead to incorrect statements about the quality and reliability of a least squares solution and the estimated parameters.

Analysis and Testing of Results

Remember that the least squares algorithm selects values for the corrections to the measurements $\hat{\mathbf{v}}$ such that the unknown parameters $\hat{\mathbf{x}}$ will have maximum probability. This is satisfied by the following condition:

$$w = \mathbf{v}^T \mathbf{V}_\mathbf{m}^{-1} \mathbf{v} \rightarrow minimum$$

To validate the reliability of the least squares solution as a whole, w should be tested to see whether it is statistically different to the degrees of freedom (r). This test is known as the *global test* or the *goodness-of-fit test*.

For the global test, w is tested using the one-tail, upper-bound χ^2 probability test at a particular level of confidence (α):

$$P\left[w > \chi^2_{r,\alpha}\right] > \alpha$$

Whilst it is theoretically possible to test w at any level of confidence, testing is commonly performed at the 95% confidence level.

Equality between w and r infers that the solution is statistically reliable. When w is significantly greater than r, in which case w will exceed the upper confidence limit, then it is inferred that the solution has failed. Reasons for failure can be attributed to one or several causes, such as blunders or outlier measurements, an incorrect stochastic model (e.g. over-optimistic variance matrix) or a shortcoming in the functional model (e.g. two or more GNSS measurements on different reference frames and/or epochs).

To assist in identifying blunders, sub-optimal variance matrices and causes of solution inconsistencies, a *local test* should be employed. For this test, two distributions may be used. Firstly, if the *a-priori* variance matrix $\mathbf{V_m}$ is known to be reliable, the magnitude of the corrections $v_1, v_2, \ldots, v_n$ should be tested against the Normal distribution with respect to a specified confidence interval (α):

$$\frac{v_i^2}{\sigma_i^2} > N(0,\alpha)$$

Secondly, if $\mathbf{V_m}$ is deemed unreliable, the magnitude of the corrections should be validated using the Student's t distribution:

$$\frac{v_i^2}{\sigma_i^2} > t(1,\alpha)$$

In either case, if measurement m_i is a blunder, outlier or the result of an error during the reduction and processing stage, v_i will be so large that this quantity will exceed the upper and lower limits of the specified confidence interval α and thereby fail the local test. Alternatively, if the specified measurement variance is too large or too small, this quantity will also fail this test.

Geodetic Network Reliability

Several statistical indicators exist for measuring the reliability of a geodetic network (Proszynski, 1994). One simple, yet useful indicator of *internal reliability* is the Pelzer (1979) reliability indicator, which may be used to quantify the reliability of individual measurements and the entire network.

The local Pelzer reliability indicator is given by:

$$\tau = \frac{\sigma_m}{\sigma_v} = \frac{\sigma_m}{\sqrt{\sigma_m^2 - \sigma_a^2}}$$

where σ_m is the *a-priori* measurement precision and σ_v is the precision of the least squares correction determined from the former and the precision of the adjusted measurement. The quantity τ ranges from unity to infinity. Good internal reliability is achieved when τ approaches unity, or when σ_v approaches σ_m. Conversely, large values of τ infer poor reliability.

The global Pelzer reliability indicator is given by:

$$T = \frac{1}{n}\sum_{i=1}^{n}\left(\tau_i^2 - 1\right)$$

where n is the number of measurements and τ_i is the local Pelzer indicator. This quantity ranges from zero to infinity and provides a basis for assessing the reliability of the network as a whole.

A General Estimation and Testing Procedure

The least squares adjustment procedure for establishing international reference frames is beyond the scope of this fact sheet. However, for the purposes of propagating regional, national or local reference frames onto other stations, the following steps may be adopted:

1. Correct all geodetic measurements for all known systematic error sources.
2. Verify that the *a-priori* measurement variances ($\mathbf{V_m}$) are realistic quantities.
3. Ensure a sufficient number of redundant measurements have been captured so as to provide a suitable level of reliability.
4. Undertake a minimally constrained adjustment (i.e. a minimum number of station coordinates are constrained sufficient to calculate all dimensions of the reference frame).
5. Test the minimally constrained adjustment using the local test and global test. Quantify local and global internal reliability.
6. Undertake a constrained adjustment by introducing all necessary reference frame station positions and uncertainties.
7. Test the constrained adjustment using the local and global tests to verify that the imposed constraints do not result in measurement failure(s). Quantify local and global internal reliability. Perform other reliability assessments as required.

In steps 5) and 7), the respective global tests should confirm that w is less than the upper limit of the desired confidence interval. Similarly, the respective local tests should confirm that the corrections to the measurements are within the upper and lower limits of the desired confidence interval. Local testing should always be performed irrespective of the outcome of the global test as it is quite possible for measurement failures to exist even when the solution passes the global test.

In the case of failures, either in the minimally constrained or constrained adjustment, the measurements variance matrix and any imposed station constraints should be examined to identify and rectify the cause of failure.

References

Cooper, M. (1974). *Control Surveys in Civil Engineering*. Collins Professional and Technical Books, London.

Mikhail, E. (1976). *Observations and Least Squares*. IEP, Dun-Donnelley, New York.

Pelzer, H. (1979). *Criteria for the reliability of geodetic networks*. *Optimisation of Design and Computation of Control Networks* (eds F. Halmos and J. Somogyi). Budapest: Akademai Kiado, pp. 553–562).

Proszynski, W. (1994). *Criteria for internal reliability of linear least squares models.* Bulletin Geodesique, vol 68, pp. 162–167.

10 GLOBAL NAVIGATION SATELLITE SYSTEMS

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Increasingly surveyors are using satellite based positioning systems to make their survey measurements. A number of systems are now fully or partially operational and several regional augmentation systems are being developed. This section provides an overview of the various systems available and methods of making measurements.

Global Navigation Satellite Systems (GNSS)

The US Global Positioning System (GPS) is the best known GNSS. "Multi-GNSS" is now understood to also refer to the other operational GNSS (Russia's GLONASS); GNSSs in various stages of deployment, such as the European Union's Galileo and China's BeiDou; and regional or augmentation satellite systems such as Japan's Quasi-Zenith Satellite System (QZSS), the Indian Regional Navigation Satellite System (IRNSS), and other space-based augmentation systems (Fig. 1).

GPS has revolutionized the disciplines of geodesy and surveying (from the early 1980s), and subsequently has made a considerable impact on all navigation communities as the availability of satellite signals and appropriate user receiver equipment has improved.

At its most basic level GPS, and in fact any GNSS, positioning mode or technique is classified according to whether it provides absolute or relative positioning results. However all require information provided by ground infrastructure.

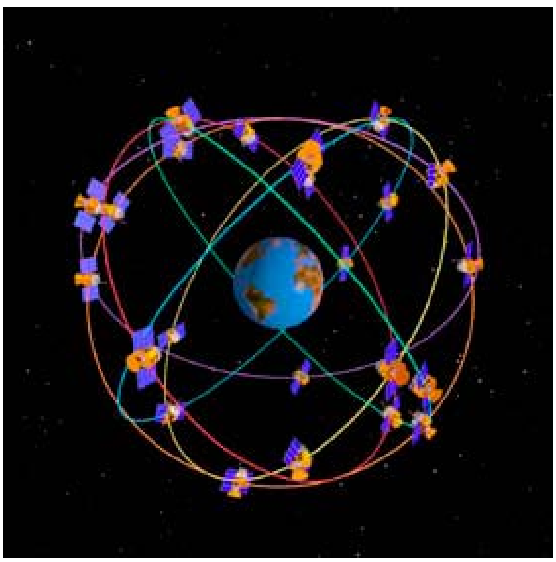

Figure 1: *Diagrammatic representation of GNSS system and satellites.*

CORS & Modes of Positioning

The ground infrastructure is today understood as the many hundreds (if not thousands) of continuously operating reference stations (CORSs). As the name implies, these are GNSS receivers (identical to surveying user equipment) which are operated at static, well-monumented stations. The highest CORS tier is that network of stations which contribute to the International GNSS Service (IGS), and in effect make up part of the physical infrastructure of the Global Geodetic Observing System (GGOS). Such CORS stations have very stable monuments, and operate continuously for many years. They contribute to the ITRF and for high-level satellite orbit and clock products to facilitate techniques such as Precise Point Positioning (PPP) (Fig. 2).

Other CORSs are operated by scientific agencies, government departments (federal, state and local), private companies, academia, and even individuals. In fact, in all cases (except for the standard GNSS Single Point Positioning mode), CORS measurements are necessary for the enhancement of GNSS positioning accuracy.

Single Point Positioning (SPP) is the operational mode for which GPS was originally designed based on comparatively low quality code or pseudo-range measurements. Standard civilian receivers currently deliver real-time, horizontal, absolute accuracy performance of the order of 5–10 m in the WGS84 reference frame. Vertical accuracy is typically 2–3 times worse than horizontal accuracy.

Differential GPS (DGPS) can overcome some of the limitations of SPP by applying corrections to the basic pseudo-range measurements at the user receiver to mitigate or eliminate some of the more serious satellite system and atmospheric biases, based on a second reference station receiver, making similar measurements at a known point. The relative positioning accuracy achievable can range from the meter-level down to a few decimeters, depending on the quality of the receivers, distance between the user

Figure 2: *Typical CORS station.*

receiver and the reference receiver generating the correction data, and the particular DGPS technique and perhaps the DGPS correction service that is used.

Relative GPS refers to the most accurate of the positioning techniques, using the DGPS principles with one or more reference stations relative to which receiver coordinates are computed from phase measurements rather than the noisier code (pseudo-range) measurements.

Real-Time Kinematic (RTK) is a relative positioning technique that can achieve centimeter-level accuracy in real-time, using pairs of receivers, even if the user receiver is moving, i.e. kinematic. Operational efficiencies and high accuracy are assured when both code and phase measurements are made on several transmitting frequencies. Hence RTK capable receivers are expensive, compared to single-frequency SPP/DGPS receivers, requiring dual-frequency instrumentation and specialized phase baseline processing algorithms.

Precise Point Positioning (PPP) is a processing technique which applies very accurate GPS satellite orbit and clock information computed separately from global CORS networks to a single high-quality receiver via specific processing algorithms, in order to produce decimeter to cm-level coordinates without any baseline constraints. Coordinates are computed in the datum of the satellite orbit and clock products, typically IGS08. Some PPP modes use local CORS network data, and are in effect relative positioning techniques.

Multi-GNSS

By the year 2020 it is expected that the number of GNSS satellites broadcasting navigation signals will double from the current number of fewer than 70 satellites, to over 140 satellites. Extra satellites and signals can improve accuracy, thus:

- More satellites being observed means a given level of accuracy can be achieved faster.
- More signals means more measurements can be processed.
- Position accuracy is less susceptible to the influence of satellite geometry.
- Vertical accuracy would approach the performance of horizontal positioning.
- The effects of multipath and interference/jamming could be mitigated through implementation of receiver autonomous integrity monitoring (RAIM) type satellite signal selection algorithms.

Extra satellites and signals can improve efficiency. For phase-based positioning to centimeter accuracy, extra satellites and signals will significantly reduce the time required to resolve carrier phase ambiguities. Extra satellites and signals can enhance signal availability at a particular location, crucial for user solutions in areas that do not satisfy the open-sky conditions.

Extra satellites and signals can improve reliability in the following ways:

- Extra measurements increase the data redundancy which helps identify measurement outliers.
- Dual-frequency operation will be enhanced in future with higher quality signals.

- More signals means that service is not as easily denied due to interference or jamming of one frequency or set of signals.

Further Information

- IGS -- International GNSS Service
  http://www.igs.org
- U.S. Government information about GPS and related topics
  http://www.gps.gov/
- European Space Agency – Galileo
  http://www.esa.int/Our_Activities/Navigation/The_future_-_Galileo/What_is_Galileo
- What is BeiDou
  http://en.beidou.gov.cn/
  http://www.spirent.com/Positioning-and-Navigation/What_is_Beidou-Compass
- GLONASS Information-Analytical Centre
  http://www.glonass-center.ru/en/index.php
- Indian Regional Navigational Satellite System (IRNSS)-1
  http://dos.gov.in/satellites.aspx

11 GNSS CORS NETWORKS AND LINKING TO ITRF

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More and more countries are building networks of Continuously Operating GNSS Reference Stations (CORS). These provide services to the surveyor that can increase the efficiency with which surveys can be undertaken. Real time positions may be generated in terms of local or global reference frames if required. This section details how to link information from a CORS to global reference frames.

How to Link CORS to ITRF

Global Navigation Satellite System (GNSS) baseline processing, whether in post processing or in real time mode, resolves a baseline between a known reference and an unknown rover position. The baseline computation is often carried out in a reference frame that is mathematically linked to the International Terrestrial Reference Frame (ITRF) or the 'World Geodetic System 1984' (WGS-84) coordinate systems. In either situation, the positional coordinates of the rover are determined relative to the coordinates of the reference. Any transformation to local coordinates (or geodetic datum) is generally performed afterwards.

For effective baseline processing, the coordinates of the reference position should be known accurately. As the fundamental purpose of GNSS reference stations or Continuously Operating Reference Stations (CORS) is to provide the geospatial framework for determining the positions of rover receivers, it follows that accurate coordinates are needed for such reference stations. To facilitate this need, there is a global network of over 400 IGS (International GNSS Service) stations, whose station coordinates are expressed in terms of the ITRF.

For simplicity it could be assumed that recently derived WGS84 co-ordinates of reference stations are ITRF coordinates, as the WGS reference frame is very similar to ITRF in specification. In fact, the latest alignment and realization with ITRF was released on February 8, 2012 and is known as WGS84 (G1674).

Many countries already have a national network of GNSS CORS. Such networks will usually have been connected to the IGS network and thus the stations will have accurate ITRF coordinates. When establishing a new stand-alone reference station or network of stations, it is preferable to connect the new station or network to the national network. It is therefore recommended that the national survey organization be contacted for data from the national CORS. If there is no suitable national network, then the alternative is to connect the new station or network to the IGS network. For general information on the IGS network and how to obtain RINEX data files consult the 'Further Information' section.

To determine the ITRF coordinates of a new reference station or network, the following is suggested:

- Select one station of the new network as the 'master'.
- At the master log data (30 second or 1 minute rate is sufficient) for at least 24 hours, and ideally for up to 7 days.
- Download RINEX data from the closest IGS station or if available from the closest national station.
- Process the baseline from the IGS, or national station to the master using 24 hours of data. There are on-line services, such as AUSPOS, that will process the data. If data has been logged for several days, process the baseline for each 24-hour period and take the mean.
- If possible, process baselines from the master to an additional one or two IGS or national stations and take the mean.
- This will provide very accurate ITRF coordinates for the master and link the master to the IGS or national network.

The ITRF coordinates of the stations of the new network have to be fiducially accurate. That is, the network has to have a very high degree of precision among the stations. To achieve this, the following procedure is recommended:

- At all stations log data (30 second or 1 minute rate is sufficient) for at least 24 hours, and possibly for several days.
- Using the ITRF coordinates of the master as the starting point, process all the baselines of the network using 24 hours of data. Possibly repeat for another one or two 24-hour periods. Adjust the network if adjustment software is available.
- This will provide ITRF coordinates for all stations. The coordinates will have a high degree of relative accuracy.

Note – If real time kinematic (RTK) or differential GNSS is being used to determine coordinates of new points then the resulting position will normally be based on the reference frame defining the reference stations transmitting the real time corrections.

In a GNSS CORS network the surveyor will normally derive a height based on the reference ellipsoid, for example Geodetic Reference System 1980 (GRS80). Most users however are working with 'physical' heights based on a local height datum (such as local mean sea level) and thus need to relate the derived ellipsoid height to this local height datum. This is achieved by using a geoid model for the subject survey area.

On-Line GNSS Processing Services

For situations where there are no existing ITRF based networks, and processing software is unavailable or not suitable, there are internet based on-line processing services which derive ITRF coordinates or positions if sufficient RINEX data has been submitted for processing. These systems will provide a position solution based on an ITRF coordinate system by calculating baselines from nearby GNSS reference stations of known ITRF position. These reference stations could be in another country and / or IGS sites.

Refer to the following web locations to find out more about such on line GNSS processing services and their requirements –

- Auto Gipsy (JPL) – Service provided by JPL
  http://apps.gdgps.net/
- AUSPOS (Geoscience Australia) – Service provided by Geoscience Australia – http://www.ga.gov.au/geodesy/sgc/wwwgps/
- OPUS – Service provided by NGS, USA
  http://www.ngs.noaa.gov/OPUS/
- SCOUT (SOPAC) – Service provided by SOPAC, USA
  http://sopac.ucsd.edu/cgibin/SCOUT.cgi
- CSRS-PPP (NRCAN GSD) – Service provided by Natural Resources, Canada
  http://www.geod.nrcan.gc.ca/ppp_e.php

Further Information

The following web pages will provide more information on this topic -

- UNAVCO (For TEQC tool)
  http://www.unavco.org/software/geodetic-utilities/geodetic-utilities.html
- IGS (International GNSS Service)
  http://igscb.jpl.nasa.gov/

For information on the IGS network

- EUREF (RINEX data)
  http://www.epncb.oma.be/
- SOPAC (info on IGS and RINEX data files)
  http://sopac.ucsd.edu/cgibin/dbShowArraySitesMap.cgi
- NGS (For information on RINEX)
  http://www.ngs.noaa.gov/CORS/
- RINEX (Receiver Independent Exchange Format)
  http://www.aiubdownload.unibe.ch/rinex/rinex300.pdf
- ITRF International Terrestrial Reference Frame
  http://itrf.ensg.ign.fr/

Information for this fact sheet was sourced from 'GPS Reference Stations and Networks – An Introductory Guide' – Leica Geosystems, Switzerland 2005.

12 THE INTERNATIONAL GNSS SERVICE (IGS)

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Many of the commercially available GNSS software packages have options which enable the surveyor to download and utilize the IGS data and products in their processing. This enables the generation of precise coordinates aligned to the latest version of ITRF, which can then be transformed to a local coordinate system if desired. This section details the structure of the IGS and services provided by them.

Introduction

The International GNSS Service (IGS), formerly the International GPS Service, is a voluntary federation of more than 200 worldwide agencies that pool resources and permanent GPS & GLONASS station data to generate precise GPS & GLONASS products. The IGS is committed to providing the highest quality data and products as the standard for Global Navigation Satellite Systems (GNSS) in support of Earth science research, multidisciplinary applications, and education. Currently the IGS includes two GNSS, GPS and GLONASS, and intends to incorporate future GNSS (such as GALILEO, BEIDOU).

The Service provides freely-available data and data products for the scientific and surveying communities. It is a service of the International Association of Geodesy (IAG) and is a collaborative effort involving dozens of scientific, academic and government organizations. Utilizing the data and products of the IGS enables high-precision positioning to be carried out over large areas in terms of the International Terrestrial Reference Frame (ITRF).

Formation

High precision GNSS surveying typically involves relative positioning to connect a new station to an existing station which has coordinates in terms of the desired reference frame. For studies that are regional or global in nature, connecting to the current international reference frame is highly desirable. In the late 1980s and early 1990s, ensuring the appropriate simultaneous GNSS occupations to connect to the international reference frame was a substantial logistical challenge. Relevant marks were often distant and challenging to access, and accessing data from other agencies making GNSS observations at the same time as the campaign could be difficult. Data processing to achieve the highest levels of precision could also be challenging. Most users did not have the capability to compute their own high-precision orbits, for example.

It was apparent that the international positioning community would benefit greatly from easier access to GNSS data and the data products commonly used to process it. The use of a standardized set of data and products would also ensure that positioning could be carried out consistently across the globe, a matter of considerable importance in many scientific and surveying applications. Thus in 1994 the IGS began its activities as a service of the IAG. It was initially called the International GPS Service, later becoming the International GNSS Service to reflect the emergence of GLONASS and other GNSS.

Structure

There are five main operational components to the IGS: data providers, data centers, analysis centers, working groups and the central bureau. These are described in the table below:

Component	Description
Data Provider	Operates a GNSS station and provides the data to an IGS data center. Station operators are typically national geodetic agencies or geodetic research institutions.
Data Center	Stores GNSS data and IGS products, making these available to the positioning community.
Analysis Center	Produces the IGS products. Contributions from each analysis center are combined by the Analysis Center Coordinator to form the official IGS products.
Working Group	Works on a particular area of interest to the IGS. Examples include antennas, ionosphere and troposphere, reference frames and clock products.
Central Bureau	Coordinates all IGS activities.

Data and Products

The primary data provided by the IGS is GPS and GLONASS (where available) code and phase measurements in the form of RINEX files for each of the stations forming the IGS network. Other data includes broadcast ephemerides (orbits) for GPS and GLONASS and station meteorological information. Updated station data is available on a daily basis.

The IGS products are derived from the data. These include precise ephemerides, satellite/receiver clock corrections, Earth Rotation Parameters (ERPs), atmospheric parameters, station coordinates and station velocities. Products are available at various latencies to reflect the requirements of different applications. Ultra-rapid products are available in near real-time, but are the least precise. Rapid products are provided within one to two days and have greater precision. The final products provide the best precision and are available within ten days to four weeks (depending on the product).

The IGS also provides antenna calibration data for antennas in its network and site logs containing station metadata.

Alignment with the ITRF

The IGS provides the GPS coordinates, velocities and ERPs that contribute to the International Terrestrial Reference Frame (ITRF). GNSS has an important role providing connections between the other space geodesy techniques (VLBI, SLR and DORIS). It also helps maintain the orientation of the ITRF.

The IGS also computes its own realization of the ITRF. This IGS realization is the reference frame in which its products are expressed. There may be small differences in coordinates between ITRF and the IGS frame, due to station-specific factors such as the use of improved antenna models in the IGS realization. The transformation between ITRF and the corresponding IGS realization is zero.

Each IGS realization is referenced by a year relating to the ITRF on which it is based. Multiple IGS realizations of the same ITRF are identified by adding or incrementing a letter. For example, the IGS realization of ITRF2008 is IGS08. The updated IGS realization of ITRF2008 is IGb08.

Further Information

Altamimi Z., Collilieux X., and Métivier L., (2011) ITRF2008: an improved solution of the International Terrestrial Reference Frame, *Journal of Geodesy*, vol. 85, page 457–473, doi:10.1007/s00190-011-0444-4

Dow, J M, Neilan R N, and Rizos, C, (2009) The International GNSS Service in a changing landscape of Global Navigation Satellite Systems, *Journal of Geodesy* vol. 83, pp. 191–198, doi:10.1007/s00190-008-0300-3

Kouba, J (2009) *A Guide to Using International GNSS Service (IGS) Products*. Available online at http://igscb.jpl.nasa.gov/igscb/resource/pubs/UsingIGSProductsVer21.pdf. Accessed 7 Aug 2013.

IGS (2013) *International GNSS Service (formerly the International GPS Service)*. Available online at http://www.igs.org/index.html. Accessed 7 Aug 2013.

13-1 STANDARDS AND QUALITY OF TERRESTRIAL REFERENCE FRAMES

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Surveyors as professionals must fulfill certain legal, regulatory and/or accuracy requirements for their clients. Typically they will strive to do this in an optimal cost effective way and with the most appropriate equipment for the job at hand. Naturally this requires a good understanding and assurance in the instrumentation employed. Clients want the most from what they pay for. Legislative authorities as well as private and public companies require confidence that the services rendered are in conformity with globally accepted best practice rules. Using internationally recognized standards such as the ISO series and ensuring traceability in measurement are two internationally and widely accepted ways of doing this.

Quality and Standards

Organizations are created to fulfill a societal need. They succeed when they satisfy the needs, requirements and expectations of their stakeholders. Who are these stakeholders? They include people or organizations that have freedom to provide or withdraw something from an enterprise. They include government, suppliers, society, employees, customers, etc.

The customer is a special stakeholder. The customer is the person, or organization that receives a product or service – the one who pays. Only the customer can decide if products or services are satisfactory. Customers require quality products and services delivered on time and at a price that reflects value for money.

What are characteristics of quality? Quality products are reliable, functional, durable, secure, available, and traceable. Quality services reflect competence, responsiveness, integrity, reliability, credibility. Quality is the degree to which a set of inherent characteristics fulfills a set of requirements: a requirement being a need or expectation that is stated, generally implied or obligatory.

One generally accepted way to ensure quality is by working to recognized quality management standards. These standards provide a benchmark for products and services and a best practice model to manage processes. A standard is a rule or requirement that is determined by a consensus opinion of users. It prescribes the accepted and (theoretically) the best criteria for a product, process, test, or procedure. The benefits of a standard are safety, quality, interchangeability of parts or systems, and international consistency.

Standards have existed for thousands of years. For example, the first long distance roads in Europe were built by Imperial Rome for the benefit of their legions. The ruts created by the Roman chariots were then used by all other wagons. These later became a gauge for laying the first railway lines.

The **ISO 9000** series is the best known set of standards to measure a management system against.

– ISO is the largest developer and publisher of International Standards,

– ISO is a network of the national standards institutes of 162 countries,

– ISO is an NGO that forms a bridge between the public and private sectors,

– ISO enables a consensus on solutions for business and society.

ISO 9000 is a family of quality management system standards designed to help organizations ensure they meet the needs of customers and other stakeholders. They represent an international consensus on good quality management practices. The ISO 9000 family comprises:

– **ISO 9000** Quality Management Systems fundamentals and vocabulary installation and servicing

– **ISO 9001** Quality Management Systems requirements

– **ISO 9004** Quality Management Systems guidelines for performance improvement

– **ISO 19011** Guidelines on Quality and Environment Management Systems Auditing

ISO 9001:2008 provides the requirements for a quality management system, regardless of what the user organization does, its size, or whether it is in the private, or public sector. It is the standard against which organizations can be certified.

A quality management system provides the framework of processes and procedures used to achieve objectives. In the process approach for management systems recommended by ISO, a process is a set of interrelated or interacting activities which transforms inputs into outputs. A product or service is the result of a process.

Traceability, Calibration and Verification

ISO 9001:2008 7.6 Control of monitoring and measuring equipment stipulates:

> *The organization shall determine the monitoring and measurement needed to provide evidence of conformity to product requirements.*
>
> *The organization shall establish processes to ensure that monitoring and measurement are carried out in a manner that is consistent with requirements.*
>
> *Where necessary to ensure valid results, measuring equipment shall be calibrated or verified, or both, at specified intervals, or prior to use, against measurement standards traceable to international or national measurement standards.*

Traceability is one of the pillars of instrument calibration. All legal metrology is based on the notion of traceability. Traceability is a method of ensuring that a measurement is an accurate representation of what it is trying to measure. It ensures an unbroken chain of comparisons that ends at a national metrology institute (NMI) (Fig. 3).

A measurement comprises four parts (Fig. 1): a value, a unit, an uncertainty and a coverage factor *k*. For example a distance can be expressed as *1.02345 m* with an uncertainty *U(D)=0.0005 m* and a coverage factor *k=2*. For a normally (Gaussian) distributed variable, *k=2* represents a coverage of 2σ or roughly 95%.

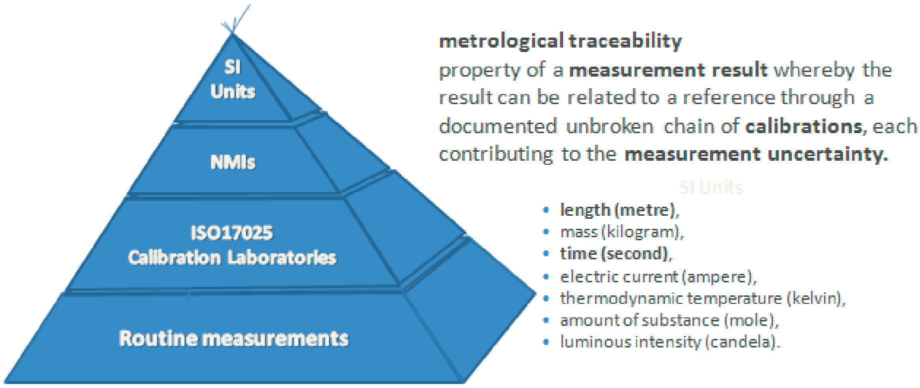


Figure 1: *The four parts comprising a measurement.*

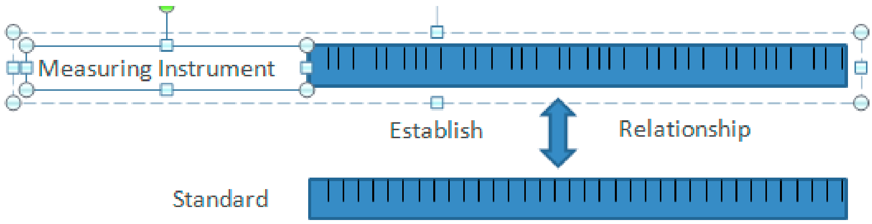


Figure 2: *Relationship between a measurement and a standard.*

The International Vocabulary of Metrology (VIM) defines calibration as:

> *operation that establishes a relation between the quantity values with measurement uncertainties provided by measurement standards and corresponding indications with associated measurement uncertainties uses this information to establish a relation for obtaining a measurement result from an indication*

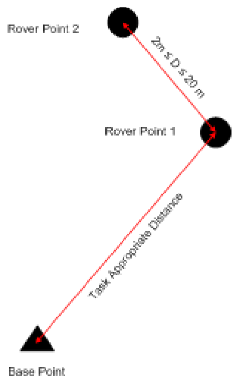
Measurement uncertainty is a non-negative parameter characterizing the dispersion of the quantity values being attributed to a measure and, based on the information used (Fig. 2). Measurement uncertainty is composed of two parts *TypeA* and *TypeB* uncertainties.

$$U = \sqrt{TypeA^2 + TypeB^2}$$

TypeA uncertainty is the result of repeated measurements. This is the uncertainty we are most familiar with. *TypeB* uncertainties include reference values, experience, manufacturer's specifications and calibration certificates.

ISO 17123 Part 8: GNSS Field Measurement Systems in Real Time Kinematic (RTK)

This standard specifies field procedures for evaluating the precision (repeatability) of Global Navigation Satellite System (GNSS) field measurement systems in real-time kinematic (GNSS RTK). These tests are intended to be field verifications of the suitability of an instrument for the application at hand, and/or to satisfy the requirements of other standards.



- Determine the distances and height differences between the two rover points to a precision of better than 3 mm
- Five sets of x, y and h coordinate measurements are made
- The difference between these measured distances ε_D and heights ε_h and those determined independently must satisfy:
  - $|\varepsilon_D| \leq 2.5 \times \sqrt{2} \times s_{xy}$
  - $|\varepsilon_h| \leq 2.5 \times \sqrt{2} \times s_h$
    - s_{xy} and s_h are a priori uncertainties
- The are two variants of this verification: the full test and the simplified test

Figure 3: *Example of traceability.*

13-2 STANDARDS AND QUALITY OF TERRESTRIAL REFERENCE FRAMES

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Section 13-1 provided information on quality and standards as they relate to reference frames. This section provides an example and further reading on the subject.

Example

Surveyors in Malaysia have a legal obligation to establish measurement traceability. This is accomplished in three steps. Reference total stations are calibrated at an ISO17025 certified laboratory at the ESRF in France. These instruments are then used to measure a series of baselines in Malaysia (Fig. 1). Surveyors check their instruments on these baselines.

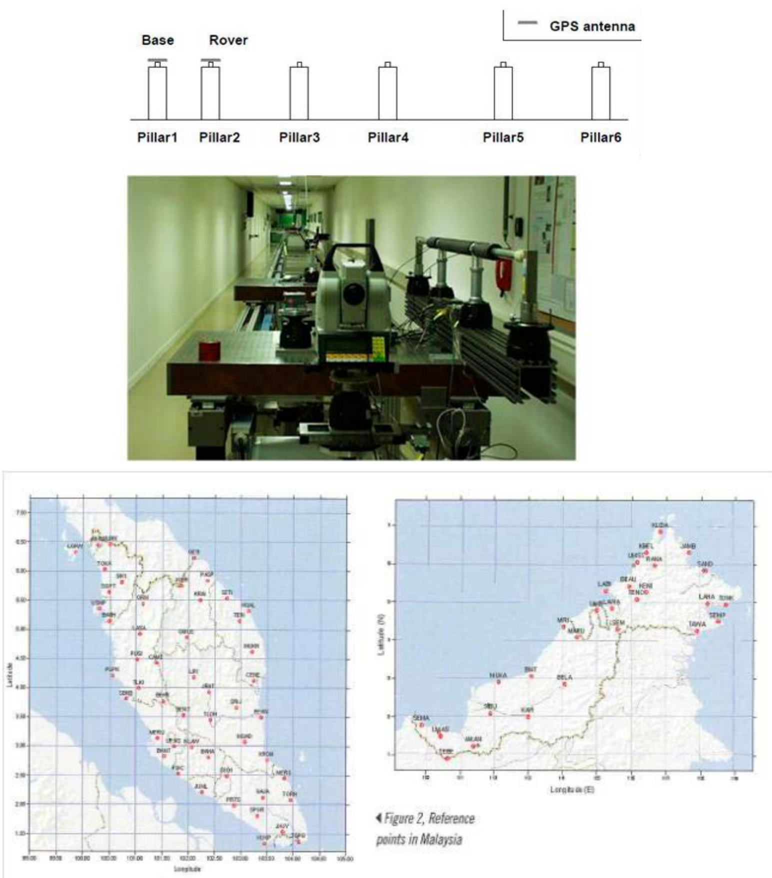


Figure1: *Example of traceability in Malaysia.*

Further Information

1. BIPM-IEC-IFCC-ISO-IUPAC-IUPAP-OIML, Ed. (1995). *Guide to the Expression of Uncertainty in Measurement*. Geneva, International Organization for Standardization.
2. Hoyle, D. (2007). *Quality Management Essentials*, Butterworth-Heinemann.
3. ISO (2001–2007). ISO 17123 Optics and optical instruments – Field procedures for testing geodetic and surveying instruments Parts 1 to 8. Geneva Switzerland, ISO. 17123.
4. ISO (2005). *ISO/CEI 17025:2005 General Requirements for the Competence of Testing and Calibration Laboratories*, International Organization for Standardization.
5. JCGM/WG2 (2007). *JCGM 200:2008: International vocabulary of metrology — Basic and general concepts and associated terms (VIM)*, Joint Committee for Guides in Metrology.
6. Martin, D. (2007). GNSS CORS – Standards and Best Practices for GNSS and CORS (*http://www.fig.net/commission5/wg52/manuals.htm*).
7. Martin, D. (2007). GNSS CORS – Calibration and Testing for GNSS and CORS V3.2 (*http://www.fig.net/commission5/wg52/manuals.htm*)
8. Martin, D. (2008). *Review of Standards, Best Practice, Testing and Calibration in Global Navigation Satellite System (GNSS)*. FIG Working Week, Stockholm, Sweden.
9. Martin, D. (2009). *GNSS CORS Calibration and Testing*. 7th FIG Regional Conference, Hanoi, Vietnam, FIG.
10. Ses, S., M. Kadir, et al. (1999). Potential use of GPS for cadastral surveys in Malaysia. *40th Aust. & 6th S.E.Asian Surveyors Congress*. Fremantle, Australia: 176–184.
11. Wübbena G., S. M., Mader G., Czopek F. (2007). *GPS Block II/IIA Satellite Antenna Testing using the Automated Absolute Field Calibration with Robot*. ION GNSS 2007, Fort Worth, Texas, USA.
12. Zeimetz, P. and H. Kuhlmann (2008). *On the Accuracy of Absolute GNSS Antenna Calibration and the Conception of a New Anechoic Chamber* FIG Working Week 2008 Stockholm, FIG.

ABOUT THE AUTHORS

Graeme Blick obtained his Bachelor of Surveying from Otago University in 1980. He worked for the then New Zealand Geological Survey (now GNS Science) in their Earth Deformation Section using geodetic techniques to measure, monitor and study crustal deformation across New Zealand. He then moved to Land Information New Zealand in 1995 where he is the Chief Geodesist for the National Geodetic Office where he continues to work on the development and implementation of the geodetic system in New Zealand, including management of its semi-dynamic datum. He is also Manager of the National Topographic Office and Business Development Group, overseeing the topographic mapping and imagery activities of the Department and new geospatial business initiatives.

Chris Crook works in the National Geodetic Office of Land Information New Zealand. He is a technical lead in the implementation of the deformation model into New Zealand Geodetic Datum 2000. He has been extensively involved in developing geodetic processes and tools, including the survey adjustment component of New Zealand's survey and title database, Landonline. Before joining Land Information New Zealand he was involved in earth deformation studies using surveying.

Nic Donnelly is a Geodetic Surveyor working in the National Geodetic Office at Land Information New Zealand. His responsibilities include assisting with the recovery of the geodetic system after significant earthquakes, developing procedures for least squares adjustments of geodetic data and managing New Zealand's semi-dynamic datum. Nic is a representative on the Australia/New Zealand Permanent Committee for Geodesy, is the FIG Special Liaison to ISO/TC211 and a member of FIG Commission 5. He is currently also a PhD student in the School of Civil and Environmental Engineering at the University of New South Wales, where his research is focused on deformation modelling and incorporating synthetic aperture radar data into a national geodetic datum.

Roger Fraser is the manager of Geodetic Survey of the Office of Surveyor-General Victoria within the Department of Transport, Planning and Local Infrastructure (Victoria, Australia). His main interests lie in the fields of reference frame geodesy, precise GNSS analysis and the development of algorithms and software for the adjustment of large geodetic networks.

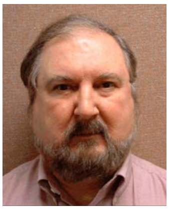
Mikael Lilje is the Head of the Geodetic Research Department at Lantmäteriet (the Swedish mapping, cadastral and land registration authority). He graduated with a M.Sc. with emphasis on geodesy and photogrammetry from the Royal Institute of Technology (Stockholm, Sweden) in 1993. He has been working at Lantmäteriet since 1994, mainly at the Geodetic Research Department. Mikael is the chair of FIG Commission 5 between 2011–14 and has been involved in FIG since 1998.

David Martin is head of the ESRF Alignment and Geodesy Group. He holds an MSc in Surveying from the University College London and a PhD in Engineering from the University of Warwick. His main interests include accelerator alignment, ground stability, hydrostatic levelling systems, and standards and traceability in instrument calibration. He is the chair of International Federation of Surveyors (FIG) Standards Network and FIG Working Group 5.1 Standards, Quality Assurance and Calibration.

Chris Rizos is Professor of Geodesy and Navigation, School of Civil & Environmental Engineering, the University of New South Wales (UNSW), Sydney, Australia. Chris is president of the International Association of Geodesy (IAG), a member of the Executive and Governing Board of the International GNSS Service (IGS), and co-chair of the Multi-GNSS Asia Steering Committee. Chris is a Fellow of the IAG, a Fellow of the Australian Institute of Navigation, a Fellow of the U.S. Institute of Navigation, and an honorary professor of Wuhan University, China. Chris has been researching the technology and applications of GPS and other navigation/positioning systems since 1985, and is an author/co-author of over 600 journal and conference papers.

Daniel R. Roman is a graduate of the Ohio State University with a Master of Science in Geodetic Science and Surveying in 1993 and a Ph.D. in Geological Sciences (emphasis in geophysics/gravity & magnetism) in 1999. After graduation, he joined the National Geodetic Survey as a Research Geodesist. He is the team lead for Geoid Modeling and Research, and his team has developed GEOID99, GEOID03, GEOID06, GEOID09, GEOID12A, and associated models. He is also the Principal Investigator for the Gravity for Redefinition of the American Vertical Datum (GRAV-D) Project. The GRAV-D Project is a US$40,000,000 project intent on developing GNSS-access geoid models for determination of cm-level accurate physical heights. The models will be developed primarily from airborne data collected across all of the U.S. states and territories and through collaboration with other countries in North America, Central America, and the Caribbean. He is currently serving as the acting Chief, Spatial Reference Systems Division at the U.S. National Geodetic Survey, responsible for maintaining access to the geometric aspects of the National Spatial Reference System.

Rob Sarib, Supervising Surveyor of the Office of the Surveyor General, with the Northern Territory Government's Department of Lands Planning and the Environment. Rob obtained his degree in Bachelor Applied Science – Survey and Mapping from Curtin University of Technology Western Australia in 1989, and also holds a Graduate Certificate in Public Sector Management received from the Flinders University of South Australia. He was registered to practice as a Licensed Surveyor in the Northern Territory, Australia in 1991. Since then he has worked in the private sector as a cadastral surveyor, and was re-employed by the Northern Territory Government to manage the Northern Territory Geospatial Reference System and the Survey Services work unit of the Office of the Surveyor General. Rob has been an active member of the FIG Commission 5 since 2002, and the Northern Territory delegate on the Intergovernmental Committee on Survey and Mapping – Permanent Committee on Geodesy. He is also a board member of the Surveyors Board of Northern Territory for Licensed or Registered surveyors.



Tomás Soler has served as an NGS scientist since 1979. Tom supervised the GPS Branch from 1990–2003 and currently serves as the Chief Technical Officer in the Spatial Reference System Division of the National Geodetic Survey. He holds a Master degree in Civil Engineering from the University of Washington at Seattle, and a Ph.D. in Geodetic Sciences from The Ohio State University. Among other scientific activities, Dr. Soler was the Chief-Editor of ASCE's Journal of Surveying Engineering for the period 2006–2013.

Richard Stanaway is the director of Quickclose, a geodetic consultancy specialising in geodetic datum analysis, transformation parameter estimation, tectonic deformation modelling and geodetic software development. After completing his surveying degree at the Queensland University of Technology, Richard was employed as an exploration surveyor in Papua New Guinea. In 2000 he densified the PNG geodetic monitoring network for the Australian National University. This work led to him undertaking a research Masters at ANU looking at the feasibility of a dynamic geodetic datum in PNG and during this time also performed maintenance of CORS in Antarctica. Richard consults to all the major mining and petroleum companies in PNG as well as the PNG Office of the Surveyor-General and advises them on geodetic matters. Since 2009, he has also been a part-time PhD research student at UNSW developing deformation models for next-generation geodetic datums and has worked closely with colleagues in New Zealand. He is the chair of the IAG working group on deformation modelling for regional reference frames and is also an active member of the FIG working group on Reference Frames in Practice.

Dr. **Neil D. Weston** is the Deputy Director of the National Geodetic Survey, NOAA. His main areas of interest are GNSS, remote sensing and 3-D image processing. Neil is a member of the International Federation of Surveyors, the American Institute of Physics and the American Association for the Advancement of Science.

FIG PUBLICATIONS

The FIG publications are divided into four categories. This should assist members and other users to identify the profile and purpose of the various publications.

FIG Policy Statements

FIG Policy Statements include political declarations and recommendations endorsed by the FIG General Assembly. They are prepared to explain FIG policies on important topics to politicians, government agencies and other decision makers, as well as surveyors and other professionals.

FIG Guides

FIG Guides are technical or managerial guidelines endorsed by the Council and recorded by the General Assembly. They are prepared to deal with topical professional issues and provide guidance for the surveying profession and relevant partners.

FIG Reports

FIG Reports are technical reports representing the outcomes from scientific meetings and Commission working groups. The reports are approved by the Council and include valuable information on specific topics of relevance to the profession, members and individual surveyors.

FIG Regulations

FIG Regulations include statutes, internal rules and work plans adopted by the FIG organisation.

List of FIG publications

For an up-to-date list of publications, please visit
www.fig.net/pub/figpub

ABOUT FIG

International Federation of Surveyors is the premier international organization representing the interests of surveyors worldwide. It is a federation of the national member associations and covers the whole range of professional fields within the global surveying community. It provides an international forum for discussion and development aiming to promote professional practice and standards.

FIG was founded in 1878 in Paris and was first known as the *Fédération Internationale des Géomètres* (FIG). This has become anglicized to the *International Federation of Surveyors* (FIG). It is a United Nations and World Bank Group recognized non-government organization (NGO), representing a membership from 120 plus countries throughout the world, and its aim is to ensure that the disciplines of surveying and all who practise them meet the needs of the markets and communities that they serve.

FIG PUBLICATION 64

The International Federation of Surveyors (FIG) Commission 5 is responsible for assisting practising surveyors in FIG member associations to apply Positioning and Measurement technologies efficiently and effectively in their day-to-day survey activities. One of the most significant technologies to emerge in recent decades has been Global Navigation Satellite Systems (GNSS). The rise of such a global technology has highlighted the need for countries to move from locally defined geodetic datums to more global datums based on the International Terrestrial Reference Frame. This FIG publication is a response by Commission 5 to this trend by bringing together a series of fact sheets to better inform surveyors about some of the key issues they need to consider as they realign and upgrade their professional knowledgebase.

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