INTER-GOVERNMENTAL COMMITTEE ON

SURVEYING AND MAPPING

STANDARDS AND PRACTICES

FOR

CONTROL SURVEYS

(SP1)

Version 1.6

Nov 2004

ICSM Publication No. 1

List of Tables

Part A

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ABOUT THIS MANUAL

The manual is subdivided into four major parts, two of which form the bulk of the material, these are:

PART A: STANDARDS OF ACCURACY, and

PART B: RECOMMENDED SURVEY AND REDUCTION PRACTICES,

The other two parts are Part C (Marking) and Part D (Data).

The purposes for which Parts A and B have been prepared are quite separate and distinct, as explained below.

PART A:

- Provides member Agencies of the Inter-Governmental Committee on Surveying and Mapping (ICSM) with clear standards of accuracy for control surveys. As such, ICSM members, at both state and federal levels, should aim to adhere to these accuracies in order to achieve uniformity of standards throughout all national and state control networks in Australia.

- The standards also provide a very convenient method by which anyone wishing to define quality control standards for a survey can simply quote the appropriate levels required, with reference to this publication.

- Standards of accuracy are independent of technique, in as much as accuracies can be achieved by differing approaches, as long as the selected methods are, in themselves, at the appropriate level of precision.

PART B

- The Recommended Survey and Reduction Practices should be viewed as a guide only, which show some techniques that can be employed to attain the necessary levels of precision needed and to achieve the accuracy standards in Part A.

- The Recommended Practices are by no means exhaustive, and are certainly not regarded as the mandatory technique necessary to achieving required accuracy outcomes; this is especially relevant in regard to the preparation and assessment of contracts.

- The choice of technique is essentially a professional decision, in which the surveyor must match the techniques employed to the results to be achieved, in order to design a homogeneous, efficient and economic survey for the project under consideration.

AMENDMENTS

The master copy of this publication is available from the ICSM World Wide Web site. This master should be checked periodically for possible updates.

Version 1.0
September 1990 - Amendment to Part B-2.4 Differential Levelling by the replacement of Pages B-4.3 and B-4.4 with new pages.

Version 1.2
August 1994

- Header sheet, to replace sheet in outer cover's pouch
- Version 1.2 title page.
- "About This Manual" page, contrasting the standards v.v. the recommended practices.
- New Contents page (i) for SP1.
- Acronym change on Foreword page (ii).
- Amendment to Glossary of Terms (page iv).
- Amended Acronyms page (viii).
- Updated "Amendments" page (ix).
- Amendment to Part A: Replacement of pages A-2, A-3, A-5, A-7, A-10, A-11 (diagram), A-13, A-16, A-17 and A-20.
- Removal of pages A-17 and B-12.2.
- Amendment to Part B: Replacement of pages B-12.1, B-13.1, B-13.2 and B-13.7.
- Replacement of Part B, Section 2.6, Global Positioning System (GPS).
- Amendments to Part D: Replacement of pages D-9, D-10, D-11, D-12, D-13 and D-14.
- Amended page, listing the ICSM member organisations (page a).
- Amendment to References (pages b and c).

Version 1.3
October 1996

Glossary of Terms	- New GDA related terms added - minor word changes
Acronyms	- New GDA related terms added
Exec Summary.	- update definition of Class - minor word modifications
Introduction	- Warning added
A-2.2	- relocate explanations of Class and Order from 2.2 2.2.1 and 2.2.2 - update definition of Class - change “Specifications” to “Practices”
A-2.2	- GDA related wording added
A-2.2.1	- additional explanation of Class - relocate definition of Class -modifications for GDA - delete reference to “minimum distance of 1 km” - minor word modifications
A-3	- Changes to Class and Order for vertical control
A-4.	- GDA related wording added - Spelling correction in table 1
A-Annex A	- “linear error.. 1.96” changed to 58.8 for Class C
A-Annex B	- spelling correction

	B-2.2 Table 1	- Amendments made.
	B-2.2.1 table 2	- Classes E added
	B-2.4	- Class L2A added & amendments made
	B-2.6.4	- Sentence deleted.
	B-2.9	- section on Transit Doppler removed (superseded technology)
	B-5.1.1 & 5.1.2	- minor word changes
	B-6.1	- amendment for GDA
	B-6.2	- amendments for GDA
	B-6.3.3, B-6.3.3.1	- minor word changes
	B-6.3.3.1.6	- explanation/warning added
	B-6.4	- amendment for GDA
	D-3.7 & 3.8	- minor word changes
	a	- ICSM contact list revised
10 March 1997	2.2.1	- Rewritten as far as table 2.2 to better explain CLASS
19 March 1997	Executive Summary(A4)	spelling correction
19 March 1997	6.3.4.1	Spelling correction
21 May 1997	Glossary	Explanation of GRS80 added
21 May 1997	various	Minor amendments to page footers.
8 July 1997	Amendments, iii	Updated ICSM WWW address
14 July 1997	Glossary	AGD66 & AGD84 Titles added
14 July 1997	a	New South Wales ICSM phone number changed.
14 July 1997		Minor amendments to page footers.
22 February 1999	B-2.6.2.1 Table 1	Replace (static method) with (classic-static method)
Version 1.4		
November 2000	Amendments, iii	Updated SP1 WWW address
	Glossary	Added GPS Glossary section Re-ordered in alphabetical order an re-formatted Definitions of AGD, GDA & WGS84 unified & updated Numerous other minor updates
	Part B Cover page	Renamed to Best Practice Guidelines for Surveys and Reductions
	Part B (2.4)	Levelling section divided into two parts: 2.4.1: Spirit, Auto or Digital Levelling (existing text, expanded by information on Digital levels & Bar coded staves) 2.4.2: EDM Height Traversing (a new technique). 'Glossary of Terms' converted to Notes at relevant Tables. Further references added to 'Recommended Reading' (now Section 2.4.3). Entries in this section abbreviated.
	Part B (2.6)	GPS section extensively modernised and expanded (in parts) by text from the ICSM 'Best Practice Guidelines - GPS', a document now superseded by SP1, version 1.4. Added guidelines for the RTK technique.
	References	Expanded by new references on GPS and, mainly, on Levelling.
	Part B (6.3.3)	Transformation options updated
	Whole document	Page numbering & style changed
	Whole document	Document converted to PDF format

Version 1.5 May 2002		
	Amendment Suggestion Sheet	Removed – replaced by an Email link on the WWW page.
	Exec Summary	Modified to include **Positional & Local Uncertainty**
	Glossary	**Positional & Local Uncertainty** added
	Acknowledgments	Added US Federal Geodetic Data Committee (FGDC)
	Section A 4	Added **Positional & Local Uncertainty**
	A 2.1	Datums – now GDA & NZGD2000 specific
	A 2.2	Added references to **Positional & Local Uncertainty**
	A2.2.3	Added examples for Class & Order for horizontal control
	A 3.2	Added references to **Positional & Local Uncertainty**
	A 3.2.3	Examples for Class & Order for Vertical control moved from previous Annex B
	A4	Absolute Positioning Techniques deleted **Positional & Local Uncertainty** added
	Annex B	Moved to the new section A 3.2.3
	B 2.6.3	Added reference to ITRF and the alignment with WGS84
	B 2.6.7	Added statement about the need for a clear sky view for GPS
	B6	Datum Transformations - replaced with a link to the GDA Technical Manual
	D 2.13	Recommended Documentation Practices – added recommendation about storing observed heights rather than derived heights.
	D 4	Data Archiving Policy - Replaced previous detail with a more generic statement
	D 5	GPS Data Elements Recommended for Archiving – rationalised for modern GPS usage.
	ICSM Contact List	Replaced with link to ICSM WWW
	References	Added reference to calculation of 95% uncertainty circle.
	Whole document	Numerous minor typographical corrections
Sept 2002	Table B 22	"Minimum Ground Clearance of Line of Sight" increased from 0.5 to 1.0 m for Class L2A, LA, LB EDM Height Traversing.
Version 1.6		
Nov 2004	2.6.3	Relationship between AHD and the geoid mentioned.
	2.6.14	Section added on GPS observations for global/regional processing
	Various	Web links updated
Dec 2007	4.1.2.1	Typo fixed in worked example (0.000038)

FOREWORD

This manual replaced the National Mapping Council (NMC) of Australia's first technical document Special Publication 1 (SP1), "Standard Specifications and Recommended Practices for Horizontal and Vertical Control Surveys".

SP1 (Standard Specifications and Recommended Practices for Horizontal and Vertical Control Surveys) was first published in April 1966. It was subsequently revised and reprinted in November 1976 and November 1981. These revisions were in general of a minor nature, being designed to introduce metric standards and a classification for Doppler surveys. Doppler was removed in version 1.3.

The NMC recognised that changing practices and technologies would impact on this document and charged the Working Party on the Impact of Technology on Geodetic Survey to report on the adequacy of SP1. This Working Party recommended that SP1 should be 'extensively revised'.

The 1986 meeting of the Technical Advisory Committee of the NMC recommended (Recommendation 24 (d)) that Council establish a working party to prepare a revised edition of SP1.

The NMC GPS Working Party also recognised some inadequacies in the SP1 and, in October 1986, reported to Council that 'the internal consistency of GPS surveys might need to be degraded when the survey is adjusted into the AGD network. This has highlighted the need to be able to separate matters relating to precision from those related to final adjustment accuracy'.

In his letter to Council members of 13th October 1986, the NMC Chairman, Mr Veenstra, formed a Working Party for the revision of SP1. Subsequent to the demise of the NMC, this Working Party continued under the direction of the Inter-Governmental Committee on Surveying and Mapping (ICSM). The work contained herein reflects the dedicated efforts of the Working Party which was constituted as follows:

Mr K Alexander	Assoc. L.S. (P.T.C.), B. App. Sc. (WAIT), L.S., M.I.S. Aust., Department of Land Administration, Western Australia
Mr G Batten	B.Tech (Surv.)(SAIT), Assoc.Dip.Sc. (N.T.Uni.), L.S., M.I.S. AusDepartment of Lands & Housing, Northern Territory
Mr C Bosloper	Ir. Geod. (Delft), M.Surv.Sc.(UNSW), Land Information Centre, Department of Lands, New South Wales
Captain P Clark	B.Surv.(UNSW), M.Surv.Sc.(UNSW), Royal Australian Survey Corps
Mr A Deck	Dip. Tech. Surv. (BCIT), B. Surv. (UNSW), Hydrographers Office, R.A.N.
Mr A Fennell	L.S., Assoc. I.S. Aust., Department of Property and Services, Victoria
Mr W Harvey MBE	B.App.Sc. (Surv. and Mapping) (WAIT), M.I.S. Aust., M.A.S. PNG, Australian Surveying and Land Information Group, Commonwealth Department of Administrative Services, Canberra.
Captain P Jensen	B.App.Sc. (Surv. and Mapping) with distinction (WAIT), M.S. (Geodetic Science Ohio State), Royal Australian Survey Corps

Mr B Murphy	L.S., M.I.S. Aust., M.A.I.C., Australian Surveying and Land Information Group, Commonwealth Department of Administrative Services, Canberra
Mr A Prichard	B.Surv. (TAS), L.S., M.I.S. Aust., M.A.I.C., Department of Environment and Planning, Tasmania
Mr G Rush	B.Surv. (U. of Q.), L.S., M.I.S. Aust., Division of Geographic Information, Department of Lands, Queensland (WORKING PARTY CONVENOR)
Mr D Warhurst	B. Tech. (Surv.) (ADEL.), L.S., M.I.S. Aust., M.A.I.C., Department of Lands, South Australia

This Working Party was assisted with contributions from all sectors of the Australian surveying and mapping industry, and thanks are extended to all those who so contributed. The Working Party also wishes to acknowledge that considerable reference was made to the publications listed in the reference section of this manual.

Members of the ICSM Geodesy Technical Sub-Committee have produced subsequent revisions to this document.

GLOSSARY OF TERMS

Accuracy	Is the degree of conformity or closeness of a measurement to the "true" value.
Australian Geodetic Datum	Is defined in NMC Special Publication 10. A synopsis of the information contained in that document follows:
AGD	The Australian Geodetic Datum (AGD) was adopted by the National Mapping Council on 21 April 1966 and proclaimed in the Commonwealth Gazette No 84 of 6 October 1966.
ANS	The AGD is defined by: the Australian National Spheroid (ANS), for which the defining parameters are: Major semi-axis (a) = 6378160 metres Flattening (f) = $^{1}/_{298.25}$ and by the coordinates of Johnston Geodetic Station being: Geodetic Latitude 25^{o} 56' 54.5515" South Geodetic Longitude 133^{o} 12' 30.0771" East Ellipsoidal Height 571.2 metres In March 1966, all geodetic surveys in Australia to that date were re-computed on the newly defined Australian Geodetic Datum.
GMA82	In October 1984, the National Mapping Council adopted the 1982 readjustment of Australian geodetic observations - Geodetic Model of Australia 1982 (GMA82) as the first step in the conversion process to a geocentric geodetic datum. The GMA82 adjustment maintained the AGD as originally defined. The coordinates derived by the GMA82 adjustment became known as AGD84. The GMA82 adjustment incorporated many Doppler baselines and is a truly ellipsoidal adjustment. Any observations, used in conjunction with the AGD84 coordinate set should first be reduced to the ANS, using the appropriate geoid - ellipsoid separation values in terms of N = +4.9 metres at Johnston Geodetic Station. To forestall any confusion the NMC adopted the following definitions: <u>Datum</u> Australian Geodetic Datum (AGD) <u>Ellipsoid</u> Australian National Spheroid (ANS)
AGD66	<u>1966 National Adjustment</u> AGD66 1966 Adopted Coordinate Set: AGD66 geographical coordinates AMG66 grid coordinates (UTM)
AGD84	<u>1982 National Adjustment</u> GMA82 1984 Adopted Coordinate Set: AGD84 geographical coordinates AMG84 grid coordinates (UTM)
Australian Height Datum	Is defined in National Mapping Council Special Publication 10 (NMC SP10). A synopsis of the information contained in that document follows:
AHD71	The Australian Height Datum (AHD) was adopted by the NMC in

May 1971 as the datum to which all vertical control for mapping and geodetic surveys is to be referred. In Tasmania, the AHD (Tasmania) was adopted as the result of an adjustment carried out in October 1983, and is referred to as AHD TAS 83.

Best Practices — Identified methods to achieve a desired CLASS of survey. Formerly known as Recommended Practices. These practices may not be the only way of achieving a specified CLASS, but reflect proven methods.

Class — Is a function of the precision of a survey network, reflecting the precision of observations as well as suitability of network design, survey methods, instruments and reduction techniques used in that survey. Preferably the CLASS is verified by an analysis of the minimally constrained least squares adjustment of the network

Confidence Region — Is a region within which the true value of a determined parameter is expected to fall. It is expressed as a percentage level of confidence with which should occur.

Geocentric Datum of Australia — The Geocentric Datum of Australia was adopted by ICSM on 28-29 November 1994 and was proclaimed in the Commonwealth of Australia Gazette No. 35, on the 6th September 1995

GDA94 — The Geocentric Datum of Australia (GDA) is realised by the coordinates of the Australian Fiducial Network (AFN) geodetic stations, referred to the GRS80 and determined within the International Earth Rotation Service Terrestrial Reference Frame 1992 (ITRF92) at the epoch of 1994.0.

To forestall any confusion, the following definitions have been adopted:

GDA Datum: Geocentric Datum of Australia

GRS80 — GRS80 Ellipsoid: Geocentric Reference System 1980

$a = 6378137$ metres $f = {}^{1}/_{298.257222101}$

Adopted Coordinate Set:

GDA94 Geographical coordinates

MGA94 — MGA94 Grid coordinates (UTM)

GMA82 — See Australian Geodetic Datum

GPS — Global Positioning System. See GPS Glossary at the end of this section

Internal Consistency and Repeatability — Are terms used to denote precision which is the degree of conformity or closeness of repeated measurements of the same quantity to each other

Local Uncertainty — the average measure, in metres at the 95% confidence level, of the relative uncertainty of the coordinates, or height, of a point(s), with respect to adjacent points in the defined frame.

Minimally Constrained Adjustment — A minimally constrained adjustment is a least squares adjustment of all observations in a network with attached a-priori variances where the only constraints are those necessary to achieve a solution. For example:

Triangulation Network: +coordinates for one station, plus one distance and one azimuth.

Vertical Control Network: height of one station only

Order — Is a function of the CLASS of a survey, the conformity of the new survey data with an existing network coordinate set AND the precision of any transformation process required to convert results from one datum to another

Positional Uncertainty — The uncertainty of the coordinates or height of a point, in metres, at the 95% confidence level, with respect to the defined reference frame.

Standards	Define the levels of precision and accuracy, which need to be achieved to meet a prescribed CLASS or ORDER
UTM	Universal Transverse Mercator projection of latitude and longitude
Variance Ratio Test	Is a method for assessing the quality of the data used in a least squares adjustment. (See Annex A in Part A for more details).
World Geodetic System 1972	WGS72 is a geocentric (earth-centred) coordinate system, which has been superseded by WGS84.
World Geodetic System 1984	The WGS84 is the geocentric datum now used for broadcast and precise ephemerides associated with GPS satellite systems. This datum is realised by the WGS84 coordinates of a number of world-wide tracking stations (maintained by the US Defence Agency) and referred to the WGS84 ellipsoid. The defining parameters of the WGS 84 ellipsoid are: Semi-major axis (a) = 6378137 m Flattening (f) = $^{1}/_{298.257223563}$

GPS GLOSSARY

Ambiguity Resolution	With carrier phase observations, the number of carrier phase cycles between the receiver and satellite is generally unknown. This is known as the ambiguity and it is an integer number. Single and double differences are also affected by ambiguities, formed by a linear combination of carrier phase integer ambiguities, e.g. a single or double differenced ambiguity. Where the integer ambiguities are unknown, the processing software may estimate them. In some cases these real-valued estimates may be used to determine the correct integer values which are then held fixed. This is called "ambiguity resolution" and "ambiguity fixing", respectively. A float solution is derived when the real-valued estimates are used, rather than the integers.
Baseline	In baseline reduction, geodetic parameters are estimated at one station relative to another, with the receivers at both sites observing common satellites simultaneously.
Broadcast Ephemeris	The broadcast ephemeris is the predicted position of the satellite in its orbit as a function of time. This is computed from the ephemeris parameters contained in the navigation message broadcast on both the L1 and L2 carrier waves
Carrier Phase	The phase (as measured at the antenna phase centre of a GPS receiver) of the two sinusoidal radio signals (the two "carriers") that are continuously emitted by each GPS satellite.
Data Editing and Cycle Slips	Data editing is required for editing cycle slips and for data sampling. Cycle slips occur when there are breaks in the continuity of signal in a satellite-receiver pair. Data sampling requires choosing the sampling rate and the starting and finishing epochs for the observations.
Differencing	<u>Non Differencing (One-Way Phase)</u> The measured carrier phase between one satellite and one receiver. <u>Single Difference (First Difference)</u> The difference between 'one-way' measurements recorded at 2 receivers, that is, 2 receivers simultaneously observing a common satellite and differencing their recorded measurements.

	Double Difference (Second Difference) The difference between 2 single differences, that is, 2 stations observing 2 satellites, forming differences between the site pair and the satellite pair. Triple Difference (Double Difference Rate/Epoch Differences) The differencing of double differences between consecutive epochs.
Ionospheric Correction	The ionosphere causes a delay in the propagation of a GPS signal that can be estimated with 50% accuracy using any recognised atmospheric model. On baselines shorter than 20 kms it is mostly eliminated by relative positioning but progressively less for increasing baseline length. For greater accuracy, it can be mostly eliminated by dual frequency observations and processing.
Multipath	Multipath errors are caused when one or more reflected signals, interfering with the main signal because of their common time origin but different path lengths, are superimposed with their relative phase offsets, on the primary signal at the receiver. Cyclic perturbations of the carrier are caused by this superimposition as the various signals undergo changes in their relative phase offsets as the geometric relation between the nearby and distant reflecting surfaces and the satellite and receiver changes.
Multistation	In multi-station reduction, geodetic parameters are estimated at more than two stations using simultaneous observations.
Precise Ephemeris	The precise ephemeris is the post-processed position of the satellite in its orbit as a function of time. It is computed from data observed at tracking stations at fixed locations and is available from various global agencies.
Pseudo Range Measurement	Obtained by comparing the time signal generated by the satellite clock to that generated by the receiver clock in order to determine propagation time, and subsequently, range.
Sampling Interval and Data Rate	The sampling interval or data rate is the interval in seconds at which observations are logged to memory.
Software	Software may be classified as data-logging software, post-processing reduction software and real time processing software, where data-logging software relates to the operation of the receiver and is not field tested. Post-processing software should previously have been tested using a bench mark data set.
Satellite Segment	The receivers must be considered as operating in a total GPS System, which includes the Satellite Segment. The major components of the satellite segment influencing testing are: (i) satellite clock stability, (ii) satellite ephemeris, (iii) signal propagation
Tropospheric Correction	The troposphere causes a propagation delay of the GPS signal. This delay can be estimated using any recognised atmospheric model. It is mostly eliminated by relative positioning for short lengths and modelled for longer baselines
Widelaning	The widelane is a linear combination of the measured phases of L1 and L2, based upon the frequency difference. The widelane ambiguities can be resolved easier than the L1 and L2 ambiguities, because the resulting 0.862 m wavelength is much longer than the individual L1 and L2 wavelengths. The knowledge of the widelane ambiguity helps to solve the L1 ambiguity, after which it is a fairly

simple computation to arrive at the L2 ambiguity. This process is called widelaning.

ACRONYMS

AGD	Australian Geodetic Datum
AHD	Australian Height Datum
AMG	Australian Map Grid
ANS	Australian National Spheroid
AUSLIG	Australian Surveying and Land Information Group (now the National Mapping Division of Geoscience Australia)
BIH	Bureau International de l'Heure
C/A code	Coarse Acquisition Code
CTP	Conventional Terrestrial Pole
DMA	Defense Mapping Agency (US)
EDM	Electronic Distance Measurement
GDA	Geocentric Datum of Australia
GDOP	Geometric Dilution of Precision
GMA	Geodetic Model of Australia
GPS	Global Positioning System
GRS80	Geodetic Reference System 1980
ICSM	Inter-Governmental Committee on Surveying and Mapping
IMU	Inertial Measuring Unit
ISS	Inertial Surveying System
IERS	International Earth Rotation Service
ITRF	IERS Terrestrial Reference Frame
LMT	Local Mean Time
MGA	Map Grid of Australia
NGDB	National Geodetic Data Base
NMC	National Mapping Council
NMC SP	National Mapping Council Special Publication
OBMA	On-board Mission Adjustment
P code	Precise code
PRN	Pseudo Random Noise
RTK	Real Time Kinematic
UT	Universal Time
UTM	Universal Transverse Mercator
VRP	Vehicle Reference Point
WGS	World Geodetic System
ZUPT	Zero Velocity Update

P A R T A
STANDARDS OF ACCURACY

EXECUTIVE SUMMARY

The ongoing requirement for the development and maintenance of a nationally accepted set of technical standards and specifications for horizontal and vertical control surveys became the responsibility of the Inter-Governmental Committee on Surveying and Mapping (ICSM) when it replaced the former National Mapping Council in 1988.

The need for a complete revision of the former National Mapping Council Special Publication 1 "Standard Specifications and Recommended Practices for Horizontal and Vertical Control Surveys", had been recognised by the former National Mapping Council when it created the Working Party in October 1986. The rapid development and introduction of many new technologies since Special Publication 1 was first published dictated the urgency of this revision.

In defining a set of national standards of accuracy for horizontal and vertical control surveys, the Working Party drew heavily on experience gained overseas, especially in North America. These standards have been defined and are summarised under the following headings:

INTRODUCTION
HORIZONTAL CONTROL
VERTICAL CONTROL
POSITIONAL & LOCAL UNCERTAINTY

<u>Introduction</u>
This section defines the fundamental requirement for a set of technical standards and specifications for horizontal and vertical control surveys undertaken either independently or co-operatively at a State or Commonwealth level. The future role to be played by ICSM in the maintenance and development of these standards and specifications is emphasised. These standards and specifications must always directly relate to the national coordinate reference systems and to the control of all: scientific, geodetic, engineering, mapping, cadastral surveys and spatial elements of land/geographic information systems.

The concepts of CLASS, POSITIONAL & LOCAL UNCERTAINTY and ORDER are introduced and briefly defined. The responsibility for assigning these quantities to a control network connected to a coordinate reference system is also defined. A guide to the application of CLASS, POSITIONAL & LOCAL UNCERTAINTY and ORDER is also given in Part A.

The requirement for State/Commonwealth authorities to acquire control survey data gathered by non-ICSM agencies and to assess this data for inclusion in the National Geodetic Data Base is discussed.

<u>Horizontal control</u>
Although the concepts discussed are applicable to any coordinates, GDA94 and NZGD2000 are now adopted for use in Australia and New Zealand respectively.
In 2000, ICSM adopted POSITIONAL UNCERTAINTY and LOCAL UNCERTAINTY as new, easily understood methods of classifying the accuracy of coordinates. POSITIONAL UNCERTAINTY is a new concept which caters for positions obtained independent of the survey network (e.g. GPS results from Wide Area Differential GPS (WADGPS) or Geoscience Australia's on-line positioning service). LOCAL UNCERTAINTY is a similar to ORDER, but is more easily understood and replaces it. However, if necessary, ORDER may still be used until LOCAL UNCERTAINTY is fully implemented. CLASS is unchanged and continues to be used to classify the quality of all aspects of a survey network.

POSITIONAL and LOCAL UNCERTAINTY provide a way of directly comparing the accuracy of positions obtained by different means. They are compatible with the ISO Technical Committee 211 (Geographic Information and Geomatics) quantities of Absolute External Positioning Accuracy and Relative Positional Accuracy (WI19115). These quantities may be applied to any position, using the best estimates available, but for geodetic surveying they are computed from the appropriate error ellipses, which will continue to be archived for geodetic survey applications, as shown in Part A, section 2.

> POSITIONAL UNCERTAINTY is the uncertainty of the coordinates or height of a point, in metres at the 95% confidence level, with respect to the defined reference frame.
>
> LOCAL UNCERTAINTY is the average measure, in metres at the 95% confidence level, of the relative uncertainty of the coordinates, or height, of a point(s), with respect to adjacent points in the defined frame
>
> CLASS is a function of the precision of a survey network, reflecting the precision of observations as well as suitability of network design, survey methods, instruments and reduction techniques used in that survey. Preferably the CLASS is verified by an analysis of the minimally constrained least squares adjustment of the network.
>
> ORDER is a function of the CLASS of the survey, the conformity of the new survey data with an existing network coordinate set AND the precision of any transformation process required to convert results from one datum to another.

CLASS and ORDER use the standard confidence level (‘1σ’) as the standard for statistical testing and allocation of CLASS and ORDER. Both these quantities are a function of the distance between points, as explained in Part A Section 2. Information for higher levels of significance is given in Part A, Annex A.

<u>Vertical control surveys</u>

The issue of standards of accuracy for vertical control surveys is addressed in a manner similar to that used for horizontal control surveys in that the concepts of CLASS, ORDER and POSITIONAL and LOCAL UNCERTAINTY as already defined, apply without qualification to vertical control surveys.

Reference is made to the use of the Australian Height Datum (AHD71) or to the best available approximation to mean sea level datum in those areas where AHD71 values are not readily accessible. The standards acknowledge that there is a fundamental difference in the way in which errors propagate in heighting surveys which is dependent upon the particular technique used. When using differential levelling or similar techniques, errors propagate in proportion to the square root of the distance.

If using GPS, trigonometric levelling or similar techniques, it is generally believed that errors propagate in proportion to the distance.

Regardless of how levelling errors propagate with specific techniques, the one system of allocation of CLASS, ORDER and POSITIONAL & LOCAL UNCERTAINTY needs to be applied so that all results can be uniformly classified.

ACKNOWLEDGEMENTS

The concepts of CLASS and ORDER are based in part upon the 1978 Canadian publication "Specifications and Recommended Practices for Control Surveys and Survey Markers".

POSITIONAL & LOCAL UNCERTAINTY are based on the concepts of Network Accuracy and Local Accuracy published by the United States Federal Geographic Data Committee as part of the National Spatial Data Infrastructure (“Geospatial Positioning Accuracy Standards Part2: Standards for Geodetic Networks”).

1. INTRODUCTION

The surveying and mapping of Australia at the national level is a co-operative enterprise shared between a number of Commonwealth and State agencies. Such an enterprise demands a high degree of co-operation between the participating agencies and in the past this has been facilitated by coordination through the National Mapping Council (NMC). The NMC was replaced in 1988 by the Inter-Governmental Committee on Surveying and Mapping (ICSM) which has, as one of its functions, the requirement to maintain and develop technical standards and specifications. This document defines the standards and specifications relevant to control surveys.

Fundamental geodetic networks of horizontal and vertical control provide Australia with a national asset in the form of fixed homogeneous coordinate reference systems. These systems are inherent to the national technological infrastructure as they form the basis of all spatially related information. Control surveys which are tied to the coordinate reference system are to be assigned a CLASS commensurate with their designed and achieved precision. Individual positions should be assigned a POSITIONAL UNCERTAINTY and a LOCAL UNCERTAINTY (or ORDER) to clearly describe the accuracy of the positions. CLASS & ORDER and POSITIONAL & LOCAL UNCERAINTY are to be assigned by the Authority undertaking the survey or by appropriate Commonwealth or State Authorities where surveys are specifically designed to provide densification of the fundamental geodetic network. A guide to the application of CLASS, POSITIONAL & LOCAL UNCERTAINTY and ORDER is given in Part A. Control surveys that are tied to the coordinate reference frame are to be adjusted using suitable computer programs.

A National Geodetic Data Base (NGDB) is held and maintained on behalf of ICSM by Geoscience Australia It is considered desirable that:

- Government authorities responsible for the geodetic survey in their State or Territory endeavour to acquire information on surveys carried out by non-ICSM members which may assist in the densification of the geodetic network;

- Information acquired be assessed by the appropriate authority to determine its suitability for inclusion in the NGDB;

- Information in the NGDB be available to all users concerned with the planning and conduct of surveying or mapping projects or with the development of spatially based information systems.

Although this publication was not designed to cover specific issues of cadastral surveys, some Authorities or States may choose to refer to it for that purpose.

2. HORIZONTAL CONTROL

2.1 DATUM FOR COORDINATES

Geodetic coordinates for the Australian mainland, Tasmania and close inshore islands are to be computed using GDA94. In New Zealand NZGD2000 should be used.

Geodetic coordinates for Australian External Territories should be computed in terms of the International Terrestrial Reference Framework (ITRF) keeping in mind that GDA94 is based on ITRF92 and subsequent versions of ITRF have not changed significantly (apart from the effect of plate tectonics).

2.2 STANDARDS OF CLASS AND ORDER

In 2000, ICSM adopted POSITIONAL UNCERTAINTY and LOCAL UNCERTAINTY as new, simple methods of classifying the quality of positions. LOCAL UNCERTAINTY replaces ORDER, but if necessary, ORDER may still be used until LOCAL UNCERTAINTY is fully implemented. CLASS is unchanged and continues to be used to classify the quality of all aspects of a survey network. POSITIONAL & LOCAL UNCERTAINTY are explained in detail in Section 4.

POSITIONAL & LOCAL UNCERAINTY are given at 95% confidence, but CLASS and ORDER use the standard confidence level (1σ) as the standard for statistical testing for allocation of CLASS and ORDER. Information relevant to various levels of significance is given in Part A, Annex A.

2.2.1 Class

CLASS is a function of the planned and achieved precision of a survey network and is dependent upon the following components:

- the network design,
- the survey practices adopted,
- the equipment and instruments used, and
- the reduction techniques employed,

all of which are usually proven by the results of a successful, minimally constrained least squares network adjustment computed on the ellipsoid associated with the datum on which the observations were acquired.

The allocation of CLASS to a survey on the basis of the results of a successful minimally constrained least squares adjustment may generally be achieved by assessing whether the semi-major axis of each relative standard error ellipse or ellipsoid (ie one sigma), is less than or equal to the length of the maximum allowable semi-major axis (r) using the following formula:

$$r = c (d + 0.2)$$

Where

- r = length of maximum allowable semi-major axis in mm.
- c = an empirically derived factor represented by historically accepted precision for a particular standard of survey.
- d = distance to any station in km.

The values of c assigned to various CLASSES of survey are shown in Table 1.

Logically and by definition, CLASS is related to the survey observations. Practically however, it may be assigned to the observations, the survey network, or it may be shown with the coordinates. In other words the CLASS value may be attached to the most appropriate survey element used by the Authority's recording system, to provide such information.

Recommended Survey and Reduction Practices (Part B) documents proven survey techniques designed to achieve a specific CLASS of survey. However, the responsibility for ensuring that any practice adopted is commensurate with the CLASS of survey planned, lies with the Authority undertaking that survey. If the survey components adopted are not commensurate with the desired CLASS of survey, or if the minimally constrained adjustment fails to achieve the desired CLASS, the survey should be assigned the highest CLASS common to all of the components. Alternatively, if a part of a survey network falls below the desired CLASS, that part of the survey may be assigned an appropriate lower CLASS.

The statistical concepts of variance and confidence region are used to express the results of a minimally constrained least square network adjustment, and replace the concept of maximum anticipated error. For any such adjustment, it is imperative that the quality of the field observations and the mathematical model be tested.

This may be achieved by an analysis of the aposteriori adjustment statistics including the application of a variance ratio test. If the statistics suggest an anomaly, or if the variance ratio test fails, then the data and the model of such an ‘unsuccessful’ adjustment must be examined to find the reason. After correction, the data must be readjusted and the assessment re-applied.

Table 1 Classification of Horizontal Control Survey

CLASS	**C** (for one sigma)	**Typical applications**
3A	1	Special high precision surveys
2A	3	High precision National geodetic surveys
A	7.5	National and State geodetic surveys
B	15	Densification of geodetic survey
C	30	Survey coordination projects
D	50	Lower CLASS projects
E	100	Lower CLASS projects

Experience has shown that with most modern methods of establishing closely spaced control, the overall pattern of error propagation is not proportional to distance but rather to: the combination of instrumental and centring errors, the effects of network configuration and a host of other contributing errors - most of which defy individual identification. The errors of measurement contributing to this pattern can be divided into two groups:

1. those proportional to distance which are dominant on lines longer than one kilometre; and

2. those non-proportional to distance which are dominant on lines shorter than one kilometre.

The adoption of the formula **r = c(d + 0.2)** as one element in the determination of **CLASS** will generally provide these specifications with the flexibility necessary to accommodate survey networks containing control stations which are closely spaced, widely spaced or with variable spacing. Nevertheless, it is recognised that the nature of survey adjustment is such that it is not always possible to fully describe the results of a survey simply by considering the statistical output of the adjustment. Part of the assessment of the quality of a survey is also dependent upon a subjective analysis of both the adjustment and of the survey itself. The ultimate responsibility for the assignment of a **CLASS** to the stations of the survey network must remain within the subjective judgement of the geodesists of the relevant authority.

The relationship between the assigned values of "c" and commonly adopted confidence regions is shown in Annex A, Table 7.

A graph of the length of the maximum allowable semi-major axis against distance between any two stations ("r" against "d"), is shown in Section 2.2, Figure 1.

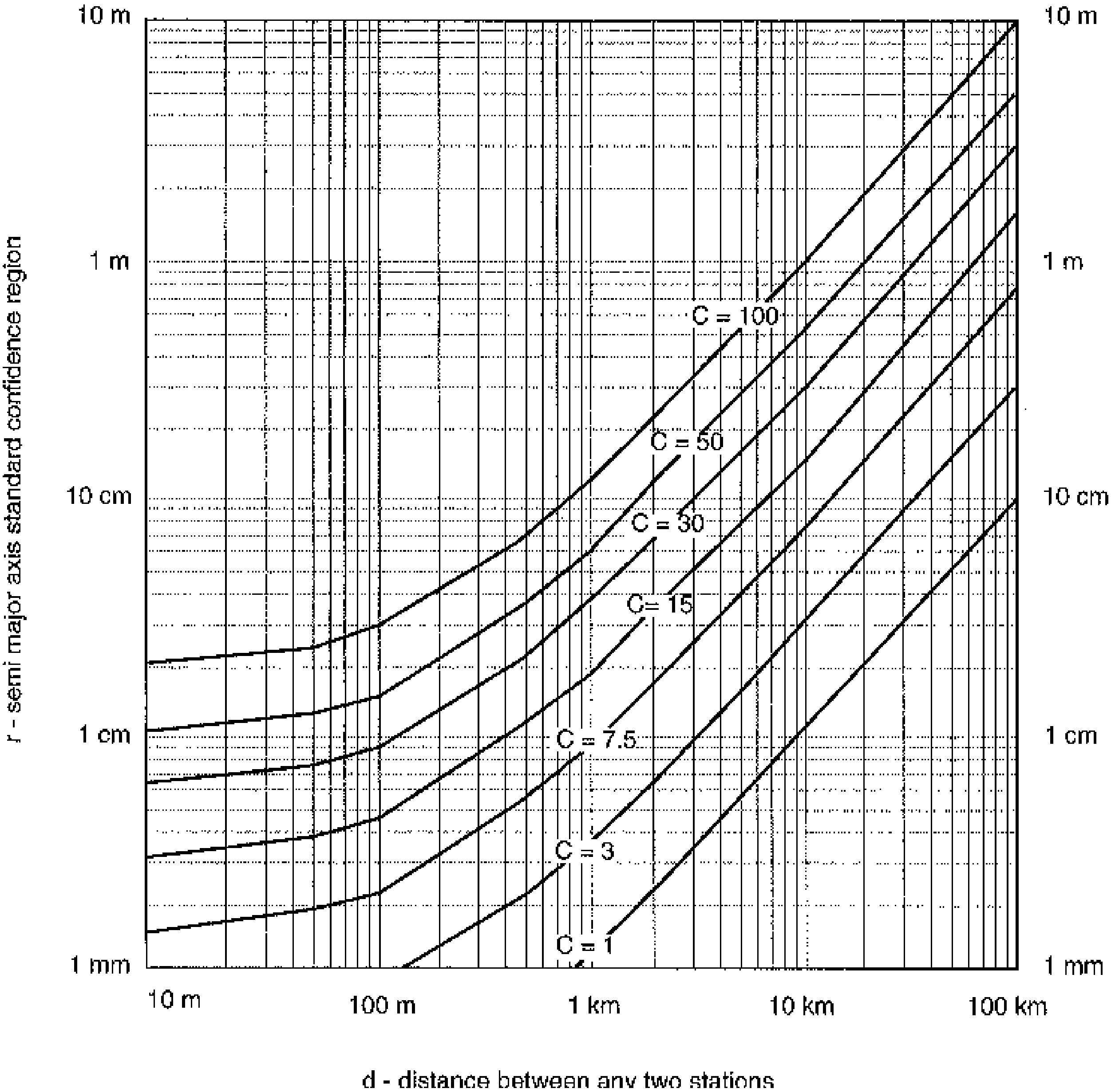


Figure 1 Horizontal Control, 2-dimensional Surveys, CLASS related values

2.2.2 Order

ORDER has been superseded by LOCAL UNCERTAINTY (see section 4), but if necessary, ORDER may still be used until LOCAL UNCERTAINTY is fully implemented.

ORDER is a function of the **CLASS** of a survey, the conformity of the new survey data with an existing network coordinate set AND the precision of any transformation process required to convert results from one datum to another.

Stations in horizontal control surveys are assigned an **ORDER** commensurate with the **CLASS** of the survey and the conformity of the survey data with the existing coordinate set.

The **ORDER** assigned to the stations in a new survey network following constraint of that network to the existing coordinate set may be;

a. not higher than the ORDER of existing stations constraining that network, and

b. not higher than the CLASS assigned to that survey.

The highest **ORDER** that may be assigned to a station from a survey of a given **CLASS** is shown in Table 2.

Table 2 Survey of a CLASS - Highest ORDER Relationship

CLASS	**ORDER**
3A	00
2A	0
A	1
B	2
C	3
D	4
E	5

As the concept of **ORDER** is based upon the CLASS of the station as well as the fit of the survey network to the existing coordinate system, the **ORDER** correlated to **CLASS** alone may be degraded by its fit to the existing coordinate set or as a result of the configuration of the ties used to constrain it to the existing system.

The allocation of **ORDER** to a station in a network, on the basis of the fit of that network to the existing coordinate set, may generally be achieved by assessing whether the semi-major axis of each relative standard error ellipse or ellipsoid, with respect to other stations in the fully constrained network, is less than or equal to the length of the maximum allowable semi-major axis. This technique is identical to that employed in the determination of **CLASS** and makes use of the same formula:

$$\mathbf{r = c (d + 0.2)}$$

Where

r = length of the maximum allowable semi-major axis in mm.

c = an empirically derived factor represented by historically accepted precision for a particular standard of survey.

d = distance to any station in km.

The values of c for various **ORDER**s of survey are shown in Table 3.

Table 3 ORDER of Horizontal Control Survey

ORDER	**C value** (for one sigma)
00	1
0	3
1	7.5
2	15
3	30
4	50
5	100

With **ORDER**, as with **CLASS**, it is recognised that assessment of the quality of a network following a constrained adjustment remains dependent upon a subjective analysis of the adjustment, the survey, and the ties to the existing coordinate system. The ultimate responsibility for the assignment of ORDER to the stations in a survey network must remain within the subjective judgement of the geodesists of the relevant authority.

2.2.3 Example - Application of CLASS and ORDER

Assigning **CLASS** to a survey.
A network of survey observations, obtained using Class A instrumentation & techniques, is adjusted in a minimally constrained least squares process which satisfies the aposteriori statistical tests.

In the adjustment output, standard (1 σ) line error ellipses (relative ellipses) are generated between adjacent points in the network. The allowable limit for the assumed Class A is calculated for each of these lines and compared to the ellipse's semi-major axis (as shown in section 2.2.1).

If all line error ellipses are less than or equal to their limit for the proposed Class, then hypothesis is true and the network may be adopted as Class A.

If all the line ellipses are greater than the limit for the proposed Class then the hypothesis is false and the network should be tested for a lower Class.

If most of the ellipses are less than the limit for the proposed Class, but a few are greater, then professional experience must be used to decide whether to downgrade the whole network, or just part of it, or to check and possibly re-observe parts of it.

e.g. for one line of a network, between points 1 and 2:

From	**To**	**Semi-major axis**	**Distance**	**Class A allowable limit**
1	2	0.23 metres	33 km	7.5(33+0.2) = 0.248 m

As 0.23 < 0.248, the assumed Class A is valid for this part of the network.

Assigning **ORDER** to a survey.
A Class A survey network is adjusted in a constrained least squares process which satisfies the aposteriori statistical tests. It is assumed that the constrained positions in the adjustment will not significantly distort the network so that the resulting positions may be classified as 1st Order.

In the adjustment output, standard (1 σ) line error ellipses (relative ellipses) are generated from each point to adjacent points in the network. The allowable limit for a 1st Order position is calculated for each of these lines and compared to the ellipse's semi-major axis (as shown in section 2.2.2).

If all the line error ellipses from a point are less than or equal to their limit for 1st Order, and the constrained points in the least squares adjustment are 1st Order or better, then hypothesis is true and the position may be adopted as 1st Order.

If line ellipses from a point are greater than the limit for 1st Order, or the constraint stations in the least squares adjustment are less than 1st Order, the hypothesis is false and the position should be tested for a lower Order.

e.g.
The line error ellipses and distances from point 1 to points 2, 3 & 4 are as shown:

From	**To**	**Semi-major axis**	**Distance**	**1st Order allowable limit**
1	2	0.23 metres	33 km	7.5(33+0.2) = 0.248 m
1	3	0.16 metres	27 km	7.5(27+0.2) = 0.204 m
1	4	0.30 metres	42 km	7.5(42+0.2) = 0.316 m

As the semi-major axes of the line error ellipses from Point 1 to Points 2,3 & 4 are all less than their respective 1st Order limit, provided all constraint stations in the constrained least squares adjustment are 1st Order or better, Point 1 may be classified as 1st Order.

3. VERTICAL CONTROL

Vertical control will normally be provided by differential levelling, trigonometrical heighting, Inertial Survey systems or by other techniques.

3.1 DATUM FOR HEIGHTS

Heights should be referred to the Australian Height Datum (AHD71). When connection to the Australian Height Datum is not possible, heights shall approximate as closely as possible to heights above mean sea level, and the datum used should be carefully defined.

Further information on the AHD can be found in The Geocentric Datum of Australia Technical Manual.

3.2 STANDARDS OF CLASS AND ORDER

In general, the definitions and assignment of **CLASS** and **ORDER** as outlined in Sections 2.2 and 2.3 are applicable for Vertical Control. In 2000, ICSM adopted POSITIONAL UNCERTAINTY and LOCAL UNCERTAINTY as new, simple methods of classifying the quality of positions. LOCAL

UNCERTAINTY replaces ORDER, but if necessary, ORDER may still be used until LOCAL UNCERTAINTY is fully implemented. CLASS is unchanged and continues to be used to classify the quality of all aspects of a survey network. POSITIONAL & LOCAL UNCERTAINTY are explained in detail in Section 4.

It is accepted that some heighting techniques (eg. differential levelling) propagate errors in proportion to the square root of the distance. Other techniques (eg. GPS and trigonometric levelling) propagate errors mainly in proportion to the distance. This is particularly apparent on distances greater than 1 km.

Therefore, different types of class and order are assigned according to the heighting technique used. The responsibility for the assignment of class and order to the heights of a survey network remains within the subjective judgement of the Authority or personnel in charge of the survey or of the vertical adjustment. A careful error analysis is particularly important when observations from both differential levelling and GPS/trig heighting are combined in a single vertical adjustment.

3.2.1 Class

Vertical control surveys are to be assigned a **CLASS** according to the planned and achieved precision. This will be a function of

- the network design,
- the survey practices adopted,
- the equipment & instruments used and
- the reduction techniques employed.

The allocation of **CLASS** to a vertical survey on the basis of the results of a successful minimally constrained least squares network adjustment may generally be achieved by assessing whether:

• **for differential levelling**	• **for GPS and trigonometric heighting**
the difference (misclose) between the forward and return levelling of a section between consecutive marks and between the end points of a level route, is less than or equal to the value (r) using the following formula:	the standard deviation of each height observation is less than or equal to the maximum allowable value (r) using the formula:
$r = c\sqrt{d}$	$r = c(d+0.2)$

Where:

r = maximum allowable error, in mm.
c = an empirically derived factor for each particular **CLASS** of survey result.
d = distance to any station in km.

The values ‘r’ for GPS/trig heighting are considered to be standard deviations.

The values ‘r’ for differential levelling are considered to be standard deviations on the condition that at least one fore-run and one back-run agree within the $c\sqrt{d}$ limits.

Table 4 values of 'c' assigned to each **CLASS** of survey

Differential levelling $r = c\sqrt{d}$		**Trigonometric and GPS heighting** $r = c(d+0.2)$	
CLASS	C (for 1 σ)	CLASS	C (for 1 σ)
L2A	2	2A	3
LA	4	A	7.5
LB	8	B	15
LC	12	C	30
LD	18	D	50
LE	36	E	100

The relationship between the assigned values of 'c' and commonly adopted confidence regions is shown in Annex A, Table 8.

A graph of the length of the maximum allowable standard deviations for GPS/trig heighting techniques is shown in Section 2.2, Figure 1. Refer to Annex B for examples of allocation of Classes.

3.2.2 Order

Similarly to the case with **CLASS**, assignment of an **ORDER** is largely technique dependent.

ORDER assigned to the height of a mark following a constrained adjustment will be commensurate with:

- the **CLASS** of the new differential levelling or trigonometric or GPS heighting,
- the order of the constraining heights,
- the precision of the transformation from one height datum to another,
- the magnitude of the discrepancy between the newly heighted and existing height differences of the survey marks at the abuttal of the new and existing levelling routes/vertical networks, and
- for GPS heighting, the accuracy of the geoid-ellipsoid separation.

Table 5 Highest ORDER which may be assigned to a height from a survey of a given CLASS

Differential levelling		**Trigonometric and GPS heighting**	
CLASS	**ORDER**	**CLASS**	**ORDER**
L2A	L0	2A	0
LA	L1	A	1
LB	L2	B	2
LC	L3	C	3
LD	L4	D	4
LE	L5	E	5

ORDER of heights from a survey is allocated on the basis of the fit of that survey to existing (constraining) heights. This technique is identical to that employed in the determination of **CLASS** and makes use of the same formulae.

Table 6 Values of c for various ORDERS of heights

Differential levelling $r = c\sqrt{d}$		**Trigonometric and GPS heighting** $r = c(d+0.2)$	
ORDER	**C (for 1 σ)**	**ORDER**	**C (for 1 σ)**
L0	2	0	3
L1	4	1	7.5
L2	8	2	15
L3	12	3	30
L4	18	4	50
L5	36	5	100

With **ORDER**, as with **CLASS**, it is recognised that assessment of the quality of heights following a constrained adjustment remains dependent on a subjective analysis of the adjustment, the survey, and ties to the existing height system. The ultimate responsibility for the assignment of **ORDER** to the heights in a survey network must remain within the subjective judgement of the Authority or personnel in charge of the survey or of the vertical adjustment.

3.2.3 Example - Application of CLASS and ORDER for Heights

Consider a closed spirit-levelling network, done using **CLASS** LC techniques in the field with appropriate data reduction and a closure of 0.007m, around a 15km long loop. The standard deviation of each of the heights assigned to the 5 newly established Benchmarks, after a minimally constrained network adjustment, is approximately 0.001m.

Despite the closure rate and height values of better than **CLASS** LA, the equipment and procedures used requires that **CLASS** LC be assigned to the network.

Alternatively had the closure rate around the loop and the standard deviation of the final assigned values of each BM been lower than **CLASS** LC, then **CLASS** LD or LE would be an appropriate allocation.

.

The same survey as above is connected into one **FIRST ORDER**, two **SECOND ORDER** and three **THIRD ORDER** existing Marks. The constrained adjustment of the levelling loop achieved better than **FIRST ORDER** agreement with the existing control. From Table 5 the highest **ORDER** to be allocated for a **CLASS** LC survey is **ORDER** L3.

Had the agreement with the existing benchmarks been to THIRD ORDER, the assigned ORDER would remain as THIRD. Should agreement have been to less than **THIRD ORDER**, the **CLASS** LC survey would be assigned a **FOURTH** or **FIFTH ORDER**, as appropriate.

All of the above assumes proper field and office checks were made on the data prior to adjustment and final allocation of **CLASS** and **ORDER**.

Alternatively, consider a spirit-levelling network observed to **CLASS** LA standards, constrained to two points with GPS/trigonometric heights of **ORDER** 4. If the two constraints are more than 1 km apart, it is likely that the spirit levelled height difference will be of higher standard. Although the levelled observations will be assigned **CLASS** LA, all heights will be allocated **ORDER** 4.

4. POSITIONAL & LOCAL UNCERTAINTY OF POSITIONS

In 2000, ICSM adopted POSITIONAL UNCERTAINTY and LOCAL UNCERTAINTY as new, simple methods of classifying the accuracy of coordinates. POSITIONAL UNCERTAINTY is a new concept that gives the quality of a position independent of the local survey network. This is particularly important when comparing positions from different sources and for positions not directly connected to the survey network (e.g. GPS results from Wide Area Differential GPS (WADGPS) or Geoscience Australia's on-line positioning service). LOCAL UNCERTAINTY is similar to ORDER and replaces it, but if necessary, ORDER may still be used until LOCAL UNCERTAINTY is fully implemented. CLASS is unchanged and continues to be used to classify the quality of all aspects of a survey network.

POSITIONAL and LOCAL UNCERTAINTY are compatible with the ISO Technical Committee 211 (Geographic Information and Geomatics - WI19115) quantities of *Absolute External Positioning Accuracy* and *Relative Positional Accuracy*. These quantities may be applied to positions determined by any means, using the best estimates available, but for geodetic surveying they are computed from the appropriate error ellipses (for horizontal positions) and standard deviations (for heights), which will continue to be archived for geodetic survey applications.

As POSITIONAL & LOCAL UNCERTAINTY are simple indicators of the quality of a position, the significant figures shown should be commensurate with their magnitude (e.g. nearest metre for values greater than 10 metres, nearest decimeter for values between 1 and 10 metres and nearest centimetre for values less than a metre).

4.1 POSITIONAL UNCERTAINTY

<u>Positional Uncertainty</u> is the uncertainty of the coordinates or height of a point, in metres, at the 95% confidence level, with respect to the defined reference frame.

- The reference frame MUST be described in the metadata. In Australia, the currently defined reference frame for horizontal positions is GDA94 and for heights is AHD. In New Zealand the currently defined reference frame for horizontal positions is NZGD2000.
- Positional Uncertainty is reported as the total uncertainty propagated from the zero order network (the AFN in Australia) or, in case of AHD heights, the total uncertainty propagated from the AHD tide gauge bench marks.

4.1.1 Formulae

4.1.1.1 Horizontal Positions

The radius of a 95% circle of uncertainty is readily calculated from the standard (1 σ) error ellipse produced by most least squares adjustment software. However, as the Positional Uncertainty is in terms of the national geodetic datum (not just the local control for a particular survey) the error ellipse used also must be in terms of the national geodetic datum. In Australia the national geodetic datum is the Geocentric Datum of Australia 1994 (GDA94) and in New Zealand it is the New Zealand Geocentric datum 2000 (NZGD2000).

In Australia, this means that the Positional Uncertainty must be calculated from a standard error ellipse that refers to the Australian Fiducial Network (AFN) or the Australian National Network (ANN) that were held fixed in the original national GDA94 adjustment. This ellipse may be obtained by one of the following methods:

1. Rigorous least squares adjustment of all observations linking the point in question back to the national geodetic datum. This would normally only be done as part of a national adjustment, in which case the error ellipses for all points in the adjustment would be available.

2. A statistical combination of the chain of error ellipses obtained from each level of the adjustment process, down to the point in question (primary, secondary, tertiary, etc). This would normally be done by the State or Territory authority responsible for their survey networks.

3. By constraining the control in the local least squares adjustment to the known error ellipse in terms of the national geodetic datum, as produced by one of the former methods.

Once the error ellipse in terms of the national geodetic datum is available, the radius of the 95% circle of uncertainty (Positional Uncertainty) can be easily calculated as shown (Leenhouts, 1985).

$C = b/a$
$K = q_0 + q_1C + q_2C^2 + q_3C^3$
Radius = aK

Where:
a = semi-major axis of the standard error ellipse
b = semi-minor axis of the standard error ellipse.
q_0 = 1.960790
q_1 = 0.004071
q_2 = 0.114276
q_3 = 0.371625

4.1.1.2 Heights
The Positional Uncertainty of a height is a linear quantity. To obtain it is simply a matter of scaling the standard deviation (1 σ) by 1.96, to convert it to 95% confidence. However, as with horizontal positions, the standard deviation of the height must be in terms of the Australian Height Datum (AHD). Currently, these values are not readily available, but will eventually be propagated from the original AHD tide gauges benchmarks.

4.1.2 Example of Positional Uncertainty
4.1.2.1 Horizontal Position

Given the dimensions of a standard error ellipse in terms of the national geodetic datum:
Semi-major axis (a) = 0.376 m
Semi-minor axis (b) = 0.035 m

Calculate
C = 0.093085
K = $1.960790 + (0.004071*0.093085) + (0.114276*0.093085^2)$
$+ (0.371625*0.093085^3)$
= 1.960790 + 0.000379 + 0.000990 + 0.000300
= 1.96

Radius = 1.962 * 0.376
= 0.74 metres

4.1.2.2 Height
Standard Deviation of a height in terms of the AHD = 0.27 m
Positional Uncertainty = 0.27 * 1.96 = 0.53 m

4.2 LOCAL UNCERTAINTY

Local Uncertainty is the average measure, in metres at the 95% confidence level, of the relative uncertainty of the coordinates of a point(s), with respect to adjacent points in the defined frame.
Each relative uncertainty used to determine this average is the uncertainty between the coordinates of two related points.
For points that are not in a common data set, Local Uncertainty is estimated by combination of the Local or Positional Uncertainties as appropriate.

4.2. 1 Formulae

The calculation of Local Uncertainty is identical to the method shown in section 4.1.1.1 for Positional Uncertainty, except that error ellipse used (or standard deviation in the case of height) is that between the two points in question, or the average of those from the point in question to adjacent points in the network.

ANNEX A - CONFIDENCE REGIONS

Background

Modern least squares adjustments of survey networks are performed on computers using a variety of software packages available. Annex A reviews the post adjustment analysis requirements of assigning **CLASS** and **ORDER** to a control network, based on the output statistics from an adjustment. It must be realised that the final **CLASS** and **ORDER** given to a survey will take into consideration other factors such as instrumentation, survey technique, ties to and precision of existing control and a number of other subjective factors.

It is necessary for adjustment software to be able to accept the survey observations and their appropriate standard deviations, variances or variance/co-variance matrices as input. Also the software should be able to output the adjustment statistics at a chosen level of confidence. The ‘1σ’ standard confidence level (one, two and three dimension) is used for the post adjustment assessment of networks in these Standards and Specifications. The test for compliance with Standards is a comparison of the semi major axes of the error ellipses/ellipsoids from the adjustment at the ‘1σ’ confidence level against the maximum error allowed. The maximum error for each **CLASS** and **ORDER** is determined from the formula **r = c(d + 0.2)** (refer PART A, Section 2.2.1) using the appropriate value of c.

Note that values of C are also tabulated for the 95% and 99% confidence level for agencies requiring statistical testing of their adjustment results at the higher levels (Refer Annex A Table 8)

Confidence Levels

In geodetic adjustments where observations are included in a mathematical model to determine one, two or three dimensional coordinates, it is usually assumed that the random errors are distributed in accordance with the Gaussian or Normal ("Bell Shaped") distribution. Observation random errors are usually considered to be linear entities, and it is expected that 68% of the observations are within one standard deviation of the mean.

As a result of the adjustment, statistics to describe the level of confidence in which the true (unknown) values of the coordinates are expected to lie with respect to the adjusted (output) coordinates, are available. In two dimensions this level of confidence or confidence region is bounded by an ellipse, and in three dimensions by an ellipsoid.

Error ellipses (2D case) may be point ellipses, which are indicators of the confidence region of the adjusted coordinates with respect to the constraining stations, or relative ellipses which indicate the precision of any station in a network relative to another station in that network. Although point ellipses may be useful indicators of accuracy, they should be assessed with care.

A network of point ellipses may not indicate the relativity between stations if there is high correlation between the stations. In a minimally constrained or constrained adjustment, there are different point ellipses for each of the stations, depending on the choice of the fixed station or stations. However in a minimally constrained adjustment, the relative ellipses are fixed. The same principles apply to error ellipsoids and relative error ellipsoids in the three dimensional case.

The standard ellipse, so called as it is scaled by the standard deviation of unit weight, contains a confidence region of about 39%. In the 3D case the standard ellipsoid contains a confidence region of about 20%. Table 7 gives the factor to be applied to the ‘1σ’ level of confidence in the 1D (68%), 2D (39%) and 3D (20%) cases. Application of this multiplying factor to the standard confidence levels of statistical output from an adjustment allows the assessment of the results at a number of different levels of confidence. These Standards and Specifications use the '1σ' confidence level as the standard on which to assess coordinate sets. The relationships between various expressions of accuracy are shown in Table 7.

Table 7 Multiplying factor used to scale adjustment output standard error statistics to various confidence levels (for 1D, 2D and 3D cases). The standard confidence level has the factor of 1.00)

MULTIPLYING FACTOR			
1D Case Standard Deviation	**2D Case Standard Ellipse**	**3D Case Standard Ellipsoid**	**DESIRED CONFIDENCE LEVEL (%)**
0.01	0.14	0.33	1
0.06	0.32	0.59	5
0.14	0.46	0.76	10
0.25	0.66	1.00	20
0.52	1.00	1.36	39
0.68	1.18	1.54	50
1.00	1.52	1.88	68
1.64	2.15	2.50	90
1.96	2.45	2.79	95
2.58	3.04	3.37	99
2.81	3.26	3.58	99.5

Table 8 Values of 'c' for various confidence levels

CLASS		**3A**	**2A**	**A**	**B**	**C**	**D**	**E**
ORDER		**00**	**0**	**1**	**2**	**3**	**4**	**5**
Dimension	**Confidence**	**CORRESPONDING c VALUE**						
1D	1σ (68%) 95% 99%	1 2 3	3 6 8	7.5 15 20	15 30 40	30 60 80	50 100 130	100 200 250
2D	1σ (39%) 95% 99%	1 2 3	3 8 9	7.5 18 23	15 35 45	30 75 90	50 125 150	100 250 300
3D	1σ (20%) 95% 99%	1 3 3	3 8 10	7.5 20 25	15 40 50	30 85 100	50 150 175	100 300 350
Error Ellipsoid (3D) 95% Confidence Region (Factor: 2.79)		2.8	8.4	20.9	41.8	83.7	139.5	279.0
Error Ellipse (2D) 95% Confidence Region (Factor: 2.45)		2.4	7.4	18.4	36.8	73.5	122.5	245.0
Linear Error (ppm) 95% Confidence Level (Factor: 1.96)		2.0	5.9	14.7	29.4	58.8	98.0	196.0
Linear Error (ppm) 68% (1σ) Confidence Level (Factor: 1.00)		1	3	7.5	15	30	50	100

NOTE:

CLASS and **ORDER** are assigned on the basis of procedures outlined in Sections 2.2.1 and 2.2.2

using the standard confidence region and the ‘c’ values listed there. However, for agencies routinely reporting at the 95% or 99% level of confidence, the appropriate ‘**c**’ values are given in Table 8 so that an adjustment doesn't need to be run twice. Provided the correct ‘c’ values are used with the corresponding confidence level of the adjustment, assigned **CLASS** and **ORDER** will remain the same whether classification is done at the ‘1σ‘, 95% or 99% confidence levels.

Procedures For Assessment Of Adjustment Statistics

The Standard confidence region (ellipse/ellipsoid) may be estimated as follows:

a. If reliable observation standard deviations are used in the adjustment, and the variance ratio test passes, the a-priori standard deviation of unit weight (normally 1) should be used to scale the standard ellipse/ellipsoid.
b. If the adjustment is distorted by constraints and the variance ratio test fails, the aposteriori standard deviation of unit weight should be used to scale the standard ellipse/ellipsoid.
c. If reliable observation standard deviations are not known, aposteriori standard deviation of unit weight should be used to scale the standard ellipse/ellipsoid.

Scaling of the output statistics appropriate to one of the three situations above can be done automatically at the time of the adjustment by choosing that scaling option available in the adjustment software. Assigning of **CLASS** and **ORDER** is then done using the procedures outlined in PART A, Sections 2.2.1 and 2.2.2.

For agencies requiring statistical testing at a higher confidence level than ‘1σ’, additional scaling of the standard ellipse is required. Testing at the higher level of confidence could be advantageous when for instance, reporting results to clients. Again, scaling of error ellipses or ellipsoids to a higher confidence level can usually be achieved automatically at the time of adjustment by selecting the appropriate options. Multiplying factors to be applied to the semi-major axes of the standard ellipse or ellipsoid are given in Table 10.

Table 9 Multiplying factors to be used for scaling the adjustment output standard error statistic to the 95% and 99% confidence level. (for various degrees of freedom, relative to one of the three cases (a), or (b) or (c) above)

Adjustment Degrees of Freedom	**Level of Confidence**					
	95%			**99%**		
	Equivalent Multiplying Factors					
	1D	**2D**	**3D**	**1D**	**2D**	**3D**
1 (c)	12.70	19.97	25.44	63.66	99.99	127.31
2 (c)	4.30	6.16	7.58	9.92	14.07	17.25
3 (c)	3.18	4.37	5.28	5.84	7.85	9.40
4 (c)	2.78	3.73	4.45	4.60	6.00	7.08
5 (c)	2.57	3.40	4.03	4.03	5.15	6.01
6 (c)	2.45	3.21	3.78	3.71	4.67	5.41
7 (c)	2.36	3.08	3.61	3.50	4.37	5.03
10 (c)	2.23	2.86	3.34	3.16	3.89	4.43
15 (c)	2.13	2.71	3.14	2.94	3.57	4.03
20 (c)	2.08	2.64	3.05	2.84	3.42	3.85
50 (c)	2.01	2.53	2.90	2.60	3.19	3.56
(a, b & c)	1.96	2.45	2.79	2.58	3.04	3.37

P A R T B

BEST PRACTICE GUIDELINES FOR SURVEYS AND REDUCTIONS

1. INTRODUCTION

Control networks are produced by making suitably accurate measurements and referring them to identifiable adjacent control points in the existing network. The combination of survey design, instrumentation, calibration procedures, observation techniques and data reduction methods comprise a control survey system.

The required **ORDER** of fit to the control points and the **CLASS** of the proposed survey will determine the field methods and reduction techniques to be employed to achieve them.

The purpose of Part B is to provide the surveyor with a guide to the minimally acceptable practices which apply to the equipment, and to the appropriate reduction methods to meet the standards of a particular **CLASS** and **ORDER** of survey.

Adherence to the Recommended Practices described in Part B is NOT mandatory in order to achieve a given CLASS. However, if not used, the onus is on the user to prove that the practices used will achieve the desired level of precision.

1.1 TRACEABILITY TO NATIONAL STANDARDS

Some of the recommended practices contained in this document take into account the legal requirement for the calibration of certain equipment in terms of the national standards of length. Other recommended practices require the regular comparison of oscillator frequencies with the National Frequency Standard. To a lesser extent, certain ancillary equipment such as thermometers and barometers must also be traceable to the national standards of temperature and pressure.

The guiding Commonwealth legislation with regard to traceability to national standards is embodied in the National Measurement Act, 1960. Supplementary requirements are embodied in relevant State legislation.

2. SURVEY TECHNIQUES

Each of the following sections deals with a specific surveying technique. The sections are not designed to be used as a text book and may not contain comprehensive lists of techniques and procedures. It is assumed the user of this document has a basic understanding of the techniques being used. If not, a suitable reference text should be consulted.

2.1 ASTRONOMICAL AZIMUTH DETERMINATION

The following table should be used as a guide to achieve results commensurate with the **CLASS** of survey required.

Table 10 Astronomical Azimuth Observation Requirement

CLASS	**A**	**B**	**C (and lower)**
Std. Dev. (single arc)	0.4”	1.5”	Range 20”
Theodolite least count	0.1”	0.2”	1”
Method	σ Octantis	Hour Angle (E&W) Ex meridian altitude of a star (E&W)	Ex meridian altitude of Sun (E&W)
Timing	1.0 sec	1.0 sec	1.0 sec
Sets	4	2	1
Arcs	6 (2 nights)	6 (2 nights)	4
Striding Level	Yes	No	No
La Place Correction	Yes	No	No
Met. Corrections	Yes	Yes	No
Pointing Interval (time)	<2 minutes	<2 minutes	<2 minutes
Altitude Range	* ± 10°	± 10°	± 10°
Azimuth Range	* Meridian ± 20°	Meridian ± 20°	Meridian ± 20°
Close Circumpolar	Yes	Optional	Optional
Elongation	Yes	Optional	Optional
Hour Angle	--	Yes	Optional
Hour Angle (sun)	--	--	Yes
Extra Meridian Altitude (Star)	--	Yes	Yes
Extra Meridian Altitude (Sun)	--	--	Yes
Vertical Bubble Calibration	Wisconsin	Optional	
Simultaneous Observations at both ends	Yes	Optional	Optional

* Not necessary or not applicable to σ Octantis in Australia.

References: Bomford 1963 [N.M. Technical Report Number 1]
Bomford 1970 [N.M. Technical Report Number 10]

2.2 ELECTRONIC DISTANCE MEASUREMENT - EDM

Table 11 EDM Observation Requirements

CLASS	**2A**	**A**	**B**	**C**	**D**	**E**
Number of days of observations	2	1	1	1	1	1
Number of sets of full measurements[2]	4	4	2	one	1	1
Move prisms between sets[4]	Yes	Yes	Yes	Optional	--	--
Range of fine readings[1]	<2(5+d)mm	<2(5+d)mm	<2(5+d)mm	7ppm	15 ppm	30 ppm
Difference between two sets[1]	<2(5+d)mm	<2.5(5+d)mm	<2.5(5+d)mm	--	--	--
Difference between means of each day's measurements[1]	< 3(5+d)mm	--	--	--	--	--
Observation between 2 hours before local noon, and 2 hours before local sunset.[5]	Yes	Yes	Yes	Optional	Optional	Optional
Atmospheric dial setting (where possible)	Zero	Zero	Zero	Optional	Optional	Optional
allow minimum warm up time.[3]	Yes	Yes	Yes	Optional	Optional	Optional

1. Where d is the length measured, in km.

2. A full measurement with a direct readout instrument shall consist of a number of readings (e.g. 6 to 10) over several minutes, after which the instrument should be re-pointed and electronically realigned, for a further group of readings. This comprises a set.
A full measurement with an indirect readout instrument shall consist of a series of fine readings on the relevant different frequencies. A set is defined as two full measurements, taken one after the other. A distance should be measured in two sets for **CLASS** B, and in four sets spread over two different days for **CLASS** 2A.

3. The minimum warm-up time to be determined during frequency determination.

4. Not required if the coarse distance is known.

5. Observations may be performed outside of the specified times (except at Sunset or Sunrise) as long as a statistically proven correction factor is applied.

Table 12 EDM Observation Requirements (continued)

CLASS	**2A**	**A**	**B**	**C**	**D & E**
Thermometer type	Mercury in glass	Mercury in glass	Mercury in glass	Mercury in glass	Mercury in glass
Graduation Interval	< 1°C	< 1°C	< 1°C	< 1° C	1 ° C
Estimate temperature to	0.1°C	0.1°C	0.1°C	0.1° C	1 ° C
Estimate pressure to	0.3 hPa	0.3 hPa	0.3 hPa	0.3 hPa	3 hPa
Wet bulb readings or relative humidity readings	Yes	Yes	Yes	Optional	--
Mets. at both ends of measured lines before and after measurements	Yes	Yes	Yes	at time of obs	--
Reciprocal vertical angles [1]	Yes simultaneous	Yes simultaneous	Yes	Optional	Optional
National standard traceability of EDM	Yes	Yes	Yes	Yes	Yes

1. Simultaneous reciprocal or reciprocal vertical angles are not required if the heights of both ends of the line are known accurately. A one way vertical angle is sufficient to determine K, the coefficient of refraction accurately.

2.2.1 Calibration Requirements

All ancillary equipment should be regularly calibrated, carry unique identifiers, and (where relevant) be regularly compared against each other.

The frequency standard should be traceable to the national standard, and calibrated once per year.

The additive constant and the oscillator frequencies of the EDM unit should be determined at least annually, and after each repair or maintenance of the EDM unit.

Table 13 Electro-optical EDM Reduction Procedures

CLASS	**2A**	**A**	**B**	**C**	**D & E**
Additive constant correction	Yes	Yes	Yes	Yes	Yes
Reflector additive constant correction	Yes	Yes	Yes	Yes	Yes
Cyclic error correction	Yes	Yes	Yes	Yes	Optional
Frequency correction	Yes	Yes	Yes	Baseline	Baseline
Barometer correction	Yes	Yes	Yes	Yes	Optional
Thermometer correction	Yes	Yes	Yes	Yes	Optional
1st velocity correction (atmospheric correction.)	Yes	Yes	Yes	Atmos. dial	Atmos. dial
Arc to chord correction (beam curvature correction.)	Yes	Yes	Yes	Over 5 km	Optional
2nd velocity correction (dip correction)	Yes	Yes	Yes	Over 5 km	Optional
Chord to chord correction (combined slope & mean sea level)	Yes	Yes	Yes	combined scale factor	Yes
2nd chord to arc correction (geoidal chord to arc correction.)	Yes	Yes	Yes	Optional	Optional
Geoid to ellipsoid correction	Yes	Yes	Yes	Optional	Optional

2.3 HORIZONTAL ANGLE MEASUREMENT

The observation requirements for horizontal angle measurement are shown in the table below. Adherence to these requirements should ensure that the appropriate level of precision is achieved.

Table 14 Horizontal Angle Observation Requirements

CLASS of Survey	**2A****	**A****	**B**	**C**	**D**	**E**
1.Required Time of Day.						
Two hours either side of sunrise/set.	Yes	Yes				
Any time except 1200-1500hrs (LMT)			Yes			
Any time, subject to checks				Yes	N/A	N/A
2.Instrument Least Count Category						
Highest	0.2”	0.2”				
High		1”	1”	1”	1”	
Medium					6”	6”
3.Horizontal Zero Settings						
Wild T3 (type)	Yes	Yes				
Wild T2 (type)		Yes	Yes	Yes	N/A	N/A

Examples of Horizontal Circle Settings for six Zero

Wild T3 (type)		00	00	05	Wild T2 (type)			00	00	10
	30	02	15			30	11	50		
	60	00	25			60	03	30		
	90	02	35			90	15	10		
	120	00	45			120	05	50		
	150	02	55			150	18	30		

4.Sets						
A. Minimum number of sets	6*	*6	2	1	1	1
B. Number of rounds per set	6	6	6	6	4	2

** Instrument and tripod should be shaded.
* Sets should be observed in equal proportion over two days.

Table 15 Horizontal Angle Observation Requirements (continued)

CLASS of Survey	2A	A	B	C	D	E
5. Field Checks						
A. Residuals from mean of any direction within each set:						
(i) should seldom exceed	3”	3”	3”	3”	5”	10”
(ii) should never exceed	4”	4”	5”	6”	10”	20”
B. Ranges within each set:						
(i) should seldom exceed	4”	6”	6”	6”	10”	15”
(ii) should never exceed	6”	8”	10”	12”	20”	30”
For applicable sets, an additional round should be observed when a range is exceeded, however if two rounds exceed the range the sets should be re-observed, under improved conditions:						
C. Ranges between sets:						
(i) should seldom exceed	1.5”	2”	3”	N/A	N/A	N/A
(ii) should never exceed	3”	4”	4”	N/A	N/A	N/A
6.Observation Corrections						
Instrumental Systematic Errors	Yes	Yes	Yes	Yes	Yes	Yes
Signal Phase Errors	Yes	Yes	Yes	Yes	Yes	Yes
Dislevelment of the Trunnion Axis	Yes	Yes	Yes	Yes		
Horizontal Refraction	Minimise using appropriate procedures for prevailing conditions.					
Deflection of the Vertical	Yes	Yes				
Skew Normals	Yes	Yes				

2.4 DIFFERENTIAL LEVELLING

Differential levelling is the conventional method of levelling for the propagation of orthometric heights. Heights are commonly propagated using spirit, automatic and digital levels. Alternatively, heights can be propagated by EDM Height Traversing, using Total Stations.

2.4.1 Spirit, Auto or Digital Levelling

Orthometric Heights are traditionally propagated by using Spirit, Automatic or Digital levels.

Table 16 Differential Levelling Equipment Characteristics

CLASS	L2A	LA	LB	LC	LD & LE
Level-minimum requirements	0.2mm/km spirit level or 0.4 mm/km digital level. (See note 2)	0.4mm/km automatic non digital level with parallel plate micrometer or 0.4mm/km digital level. (See note 2)	As for LA.	1.0-1.5 mm/km or better automatic or digital level. (See note 2)	1.5mm/km or upward (ie. less sensitive) auto-collimating or digital or spirit level.
Staff construction minimum requirements (Analog or bar coded). (See note 1)	Rigid Invar.	Rigid Invar.	Rigid Invar.	Folding staff of wood or fibreglass.	Telescopic staff of wood, fibreglass or aluminium.
Staff graduation interval (Analog staves).	5mm	5mm or 10mm	As for LA.	10mm	10mm
Tripod construction	Rigid	Rigid	Rigid	Rigid	Rigid or telescopic
Bubble attached to staff	Yes	Yes	Yes	Yes	Optional.
Solid, portable change points	No - Route is pre-marked.	Yes	Yes	Yes	Optional.
Umbrella for level	Yes	Yes	Yes	No	No

Notes on Table 17:

1. **Analog** refers to staves that have accepted metric or foot face patterns that have been developed over time for optical levels.
   Bar coded refers to staff face patterns developed specifically for digital levels.

2. **Digital Levels**. Digital levels have been developed by several manufacturers in recent years to offer automated staff reading and digital recording of levelling observations. These instruments offer protection against both staff reading errors and booking errors.

   The level utilises a compensating prism and thus many of the remarks that apply to automatic levels also apply to digital levels. The electronic sensor is capable of interpreting a bar coded staff pattern so that the instrument can record both the staff reading and the staff distance.

   Digital levels are however still capable of optical observations to analog staves at Class LC or lower.

   Different manufacturers employ different bar coding strategies so that bar coded staves are not interchangeable among instruments of different make.

   Digital levels of appropriate sensitivity (see Rueger & Brunner [2000]) are capable of Class L2A levelling when employed with suitable invar staves. Lesser sensitivity levels employed with folding or telescopic staves of wood or fibreglass are highly convenient for many lesser order purposes.

Table 17 Differential Levelling Equipment Testing

CLASS	**L2A**	**LA**	**LB**	**LC**	**LD & LE**
System test prior to commencement (eg ISO, DIN or Princeton)	Yes	Yes	Yes	Optional	Optional
Maximum standard error in the slope of the line of sight as determined by the system test	Spirit level: 1"/2mm run Automatic or digital: 0.4" setting accuracy.	Spirit level: 1.5"/2mm run. Automatic or digital: 0.4" setting accuracy.	Spirit level: 4"/2mm run. Automatic or digital: 0.8" setting accuracy.	Spirit level: 10"/2mm run. Automatic or digital: 1.0"setting accuracy.	--
Vertical collimation check (eg. Two-Peg Test) Frequency Maximum collimation error	Daily 2" or 0.3 mm over 30m. (Digital levels can "Store" the results)	Daily 2" or 0.8 mm over 80m.	Daily 4" or 1.5 mm over 80m.	Daily 10" or 4 mm over 80m.	As required 10" or 4 mm over 80m.
Level cross-hair verticality check	Yes	Yes	Yes	Yes	Optional
Staff calibration frequency	Immediately prior to commencement of levelling, and at 3 monthly intervals whilst in continued use.			Within 6 months of use.	Optional
Staff bubble verticality to be within	5' (See note 1)	10'	10'	10'	10'
Thermometers accurate to	0.5°C	1°C	1°C	1°C	Optional

Notes on Table 18:

1. 5' is equivalent to 4.5mm movement at the top of a 3m staff. Supporting braces are essential.

Table 18 Differential Levelling Observation Procedures

<table>
<tr><th>CLASS</th><th>L2A</th><th>LA</th><th>LB</th><th>LC</th><th>LD & LE</th></tr>
<tr><td>Instrument levelled by "unsystematic" method (See Note 2)</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Optional</td></tr>
<tr><td>"Leap-Frog" system of progression used (See Note 3)</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Optional</td></tr>
<tr><td>Staff readings recorded to nearest</td><td>0.01mm (see note 1). For digital levels take the mean of five with an indicated sd. of 0.0002 or less.</td><td>0.1mm. For digital levels as for L2A but with an indicated sd. of 0.001 or less.</td><td>0.1mm. For digital levels as for LA.</td><td>1mm. For digital levels as for LA.</td><td>LD 1mm
LE 10mm</td></tr>
<tr><td>Temperature recorded (When used).</td><td>Start, middle, finish and pronounced changes</td><td colspan="3">At start and finish of each levelling run and at pronounced changes of temperature</td><td>--</td></tr>
<tr><td>Maximum length sight</td><td>20-30m.</td><td>40m.</td><td>60m.</td><td>80m.</td><td>100m.</td></tr>
<tr><td>Minimum ground clearance of line of sight</td><td>0.5m.</td><td>0.5m.</td><td>0.5m.</td><td>0.3m.</td><td>0.2m.</td></tr>
<tr><td>Back-sight and fore-sight lengths to be equal within</td><td>1% (Set out by taped measurement).</td><td>1%</td><td>2%</td><td>2%</td><td>5%</td></tr>
<tr><td>Observing times (LMT)</td><td>Before 10 am & after 2 pm</td><td>Before 10 am & after 2 pm</td><td colspan="3">Any time provided atmospheric conditions allow positive resolution of staff graduation.</td></tr>
<tr><td>Two-way levelling</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td></tr>
<tr><td>Even number of instrument set-ups between bench marks.</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Optional</td></tr>
<tr><td>Minimum number of holding marks used for temporary suspension of levelling</td><td>Not to be suspended</td><td>Not to be suspended</td><td>2</td><td>2</td><td>1</td></tr>
<tr><td>Minimum number of holding marks used for temporary suspension of levelling > 5 days</td><td>Not to be suspended</td><td>Not to be suspended</td><td>overlapping marks re-levelled within
$2\sqrt{d}$</td><td>$12\sqrt{d}$</td><td>1</td></tr>
<tr><td>Maximum allowable misclosure (mm) of forward and reverse levelling runs</td><td>$2\sqrt{d}$</td><td>$4\sqrt{d}$</td><td>$8\sqrt{d}$</td><td>$12\sqrt{d}$</td><td>LD=$18\sqrt{d}$
LE=$36\sqrt{d}$</td></tr>
<tr><td></td><td colspan="5">where d is the distance in kilometres between benchmarks</td></tr>
<tr><td>Minimum number of bench marks used to prove datum</td><td>3</td><td>3</td><td>3</td><td>3</td><td>2</td></tr>
<tr><td>Datum bench marks to be double levelled</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td></tr>
<tr><td>Maximum misclose (mm) on datum bench marks</td><td>$2\sqrt{d}$</td><td>$4\sqrt{d}$</td><td>$8\sqrt{d}$</td><td>$12\sqrt{d}$</td><td>LD=$18\sqrt{d}$
LE=$36\sqrt{d}$</td></tr>
</table>

Notes on Table 19:

1. **Staff reading**: At each setup the difference between back/forward and forward/back readings must be not more than 0.1mm.

2. **Unsystematic** method of levelling instrument: When centring automatic levels with circular bubbles, the "unsystematic" method of levelling the instrument should be used whereby the telescope is pointed in forward and reverse directions at alternate set-ups, ie. always towards the same staff-man who will be "leap-frogging" each instrument set-up.

3. **Leap-frog System**: "Leap-frog" levelling involves the one staff remaining at a particular change point for both sightings. To avoid staff index error the same staff is used for the first back-sight and the last fore-sight of each levelling run.

Table 19 Differential Levelling Reduction Procedures

CLASS	**L2A**	**LA**	**LB**	**LC**	**LD &LE**
Orthometric correction to be applied	Yes	Yes	Yes	Yes	N/A

2.4.2 EDM Height Traversing

Differential levelling is the conventional method of levelling for the propagation of Orthometric Heights. A variant of the common technique of spirit levelling is EDM-Height-Traversing where the difference in height between change points is determined using observations of zenith angles and slope distances. The most convenient mode is that of Leap-Frog EDM-Height-Traversing to two reflectors of fixed height in the usual backsight / foresight mode used in levelling.

The alternative mode of (non-simultaneous) reciprocal EDM-Height-Traversing is not discussed in detail since it requires special techniques to connect to bench marks and to minimise the effects of unequal instrument and reflector heights, particularly when attempting Class L2A and LA results. See Note 2 with Table 22 EDM Height Traversing Observation Procedures.

Table 20 EDM Height Traversing Equipment Characteristics

CLASS	L2A	LA	LB	LC	LD & LE
Electronic Tacheometer (Total Station) requirements	1 mm + 1 ppm distance and 1” zenith angle	2 mm + 2 ppm distance and 1” zenith angle	2 mm + 2 ppm distance and 2” zenith angle	2 mm + 2 ppm distance and 3” zenith angle	3 mm + 2 ppm distance and 5” zenith angle
Accuracy of Level Sensor or compensator	0.5”	0.5”	1”	1.5”	2.5”
Diametrical Circle Reading on Vertical Circle (or equivalent)	Yes	Yes	Yes	Optional	Optional
Entry of Temperature and Pressure for on-line First Velocity Correction	Yes	Yes	Yes	Yes	Yes
Refraction and Earth Curvature Correction enabled	Yes	Yes	Yes	Yes	Yes
Target / Reflector construction: Minimum requirements (see Note 1)	2 Fixed Height Reflector Rods w permanently mounted, balanced and tilting Prism	2 Fixed Height Reflector Rods w permanently mounted, balanced and tilting prism	1 - 2 Fixed Height Reflector Rods w permanently mounted, balanced and tilting prism	1 - 2 Standard Reflector Rods with balanced and tilting Prism	1 - 2 Standard Reflector Rods with balanced and tilting Prism
Reflector Rod Support	Bipod / Two Leg Struts	Bipod/ Two Leg Struts	Bipod/ Two Leg Struts	Bipod/ Two Leg Struts	Optional
Bubble attached to Reflector Rod	Yes	Yes	Yes	Yes	Optional
Solid, portable change points (See Note 2)	No - Route is pre-marked	Yes	Yes	Yes	Yes
Umbrella for instrument	Yes	Yes	Yes	No	No

Notes on Table 21:

1. For Classes L2A, LA and LB the target/reflector must be securely attached to the fixed height reflector rod. If the target/reflector assembly is not permanently attached but screwed and locked into place on each day, the height of reflector must agree to 0.01 mm between multiple attachments. The reflector should be tiltable about a horizontal axis that intersects the symmetry axis well inside the prism ("balanced” prism, NO “zero error” prisms). The height of the triangular target patterns on the left and right of the prism must have the same height as the prism (to 0.01 mm) since pointing on close range (to 60 m) is to the apex of the prism and on longer range to the target.

2. All temporary (change plates) and permanently marked change points must feature a small central hole so that the reflector rod does not slide off.

Table 21 EDM Height Traversing Equipment Testing

CLASS	L2A	LA	LB	LC	LD & LE
System test prior to commencement	Yes	Yes	Yes	Optional	Optional
Calibration of index errors of vertical circle and level sensor	Daily	Daily	Daily	Daily	As required
Staff bubble verticality to be within	10’	10’	10’	10’	10’
Barometers accurate to	2 hPa	2 hPa	2 hPa	2 hPa	2 hPa
Thermometers accuracy	1°C	1°C	1°C	1°C	Optional

Table 22 EDM Height Traversing Observation Procedures

CLASS	**L2A**	**LA**	**LB**	**LC**	**LD & LE**
EDM Height Traversing Method (see Note 1)	Leap-Frog	Leap-Frog	Leap-Frog or Reciprocal (see Note 2)	Leap-Frog or Reciprocal (see Note 2)	Leap-Frog or Reciprocal (see Note 2)
Number of sets to target	2	2	1	1	1
Pointings in first set: (In second set, if appl., first FS, then BS)	BS(FL), BS(FR), BS(FR), BS(FL), FS(FL), FS(FR), FS(FR), FS(FL)	BS(FL), BS(FR), BS(FR), BS(FL), FS(FL), FS(FR), FS(FR), FS(FL)	BS(FL), BS(FR), BS(FR), BS(FL), FS(FL), FS(FR), FS(FR), FS(FL)	BS(FL), BS(FR), BS(FR), BS(FL), FS(FL), FS(FR), FS(FR), FS(FL)	BS(FL), BS(FR), BS(FR), BS(FL), FS(FL), FS(FR), FS(FR), FS(FL)
Max Spread per set	1.0 mm	1.5 mm	1.5 mm	1.5 mm	3.0 mm
Height difference recorded to nearest	0.01mm per pointing	0.1 mm per pointing	0.1 mm per pointing	1 mm per pointing	D 1mm, E 5 mm per pointing
Temperature and Pressure measured and entered into the instrument	At start, middle and finish of each 'levelling 'run and at pronounced changes of temperature				At start of 'levelling' run
Maximum length sight In Leap-Frog EDM Height Traversing	60 m	75 m	90 m	120 m	150 m
Slope distance recorded (for balancing FS and BS distances) to:	0.1 m	0.1 m	0.1 m	1.0 m	1.0 m
Minimum ground clearance of line of sight	1.0 m	1.0 m	1.0 m	0.3 m	0.2 m
Back-sight and fore-sight lengths to be equal within	1 m (set out)	2 m (set out)	5 m (set out)	10 m (set out)	20 m (set out)
Observing times	Sight lengths might have to be reduced to achieve "Max Spread per Set" in poor observing conditions (e. g. heat shimmer)				
Two-way levelling in Leap-Frog EDM Height Traversing	Yes	Yes	Yes (But NOT in reciprocal EDM-Height-Traversing)		
Even number of instrument set-ups between bench marks	Yes in Leap-Frog EDM-Height-Traversing with two reflector rods (Not applicable for Reciprocal EDM-Height-Traversing)				Optional
Minimum number of holding marks used for temporary suspension of levelling	Not to be suspended	Not to be suspended	2	2	1
Minimum number of holding marks used for temporary suspension of levelling > 5 days	Not to be suspended	Not to be suspended	3 overlapping marks re-levelled within $2\sqrt{d}$	3 overlapping marks re-levelled within $2\sqrt{d}$	1
Maximum misclosure (mm) of forward and reverse levelling runs	$2\sqrt{d}$	$4\sqrt{d}$	$8\sqrt{d}$	$12\sqrt{d}$	$D=18\sqrt{d}$ $E=36\sqrt{d}$
	where d is the distance in kilometres between benchmarks				
Minimum number of bench marks used to prove datum	3	3	3	3	2
Maximum misclose (mm) on datum BM's	$2\sqrt{d}$	$4\sqrt{d}$	$8\sqrt{d}$	$12\sqrt{d}$	$D = 18\sqrt{d}$ $E = 36\sqrt{d}$
	where d is the distance in kilometres between benchmarks				

Notes on Table 23:

1. "**Leap-Frog" EDM-Height-Traversing**: "Leap-Frog" EDM-Height-Traversing involves the one target remaining at a particular change point for both sightings. To avoid the possibility of the target being placed on a different point the target is not moved between the back-sight and foresight. Two target/reflectors are employed (on reflector rods with struts). As in spirit levelling, it is imperative that the electronic tacheometer (total station) is set up in the middle between the two reflectors. Recorded are the height differences (between the instrument's trunnion axis and the reflector) that are computed by the electronic tacheometers. In consequence, the ambient temperature and pressure must be input into the instrument since the slope distances must be corrected for temperature and pressure (first velocity correction) on-line. See Rüeger & Brunner (1982) and *The Canadian Surveyor*, 36(1): 69-87.

2. **"Non-Simultaneous Reciprocal"** EDM-Height-Traversing: Normal EDM traversing equipment is employed with one electronic tacheometer, two reflector/target assemblies and two to three tripods. To connect to bench marks, the instrument has to be set up within 20 m. The height difference between instrument and bench mark is obtained by zenith angle measurements to some marks on a levelling staff on the bench mark (or to a prism on reflector rod with struts on the bench mark). Between tripods, the zenith angles and the slope distances are measured forward and backwards. Since this provides two height differences per leg, reciprocal EDM-Height-Traversing is only done one-way. Depending on the accuracy requirements, the lengths of the legs in reciprocal EDM-Height-Traversing can be significantly longer than in Leap-Frog EDM-Height-Traversing. See Rüeger & Brunner (1982).

Table 23 EDM Height Traversing Reduction Procedures

CLASS	**L2A**	**LA**	**LB**	**LC**	**LD &LE**
Orthometric correction to be applied	Yes	Yes	Yes	Yes	N/A

2.4.3 Recommended Reading

Various papers exist on the vagaries and practical details of levelling and can be found in the references at the end of this manual. For example: Becker (1985), Becker et al (1994), NSW Surveyor General's Directions (1994), Rueger (1997, 1998, 1995, 1999), Rueger & Brunner (1981, 1982, 2000), *Surveying and Land Information Systems,* 55(4), *The Canadian Surveyor, 36(1), The Australian Surveyor*, 30(6), 40(4).

2.5 TRIGONOMETRIC HEIGHTING

Trigonometric heighting is achieved using several individual items of survey equipment. Unless directly specified to achieve a desired **CLASS** of trigonometrical heighting, use procedures and standards for the particular observation type (eg vertical angle, distance) as set out elsewhere in Part B.

Table 24 Trigonometric Heighting Observation Requirements

CLASS	**A**	**B**	**C**
Simultaneous reciprocal	Yes	Optional	Optional
Non-simultaneous Reciprocal		Yes	Optional
One way Observations			Yes
Observing times (LMT) d > 16km d < 16km	 1400-1600 1000-1600	 1400-1600 1000-1600	 1400-1600 1000-1600
Number of sets	2	2	1
Number of pointings (per set)	6	6	6
Maximum range per set	6”	6”	8”
Meteorological Observations	Yes	Yes	Yes

2.6 GLOBAL POSITIONING SYSTEM (GPS)

2.6.1 Introduction

- Over recent years the surveying profession has witnessed the growing capability and widespread use of the Global Positioning System (GPS) for a number of surveying operations.
- These guidelines indicate best practice for the use of GPS for surveying applications in Australia and New Zealand. The purpose of this section is to present these principles in general terms so that they can be applied by users to achieve a quality result.
- It should be noted that these guidelines do not represent legal traceability of measurement.
- Legal traceability of measurement for GPS is not required in New Zealand.
- Individual Government Departments of the Australian States/Territories and New Zealand may have additional requirements and may issue supplementary advice.
- If you perceive that amendments to these guidelines are required, please advise ICSM, your ICSM member, the relevant State Government Department or visit the ICSM Web site.

2.6.2 Limitations

These guidelines are specific to utilising the Global Positioning System (GPS) in circumstances that follow a quality assurance approach. The following understandings and limitations therefore apply:

- These guidelines apply to GPS hardware and software systems designed for survey applications operated in differential mode where GPS carrier phase and pseudorange observations are recorded by the GPS receivers.
- These guidelines should be read in conjunction with:
  - Part A of this document (Standards of Accuracy) and
  - For New Zealand applications, Department of Survey and Land Information (DOSLI) Survey System Immediate Report 96/1, New Zealand Standards of Accuracy for Geodetic Surveys, May 1996. (Note: The Department of Survey and Land Information became Land Information New Zealand from 1 July 1996).

- Existing general standards, specification procedures and practices for marking, mark verification, density of control, numbering, redundancy of observations, closure, connections, lodgement of work, etc, still apply, as set out in various Australian State/Territory and New Zealand Survey Practice Directions and Regulations, and other relevant legislation.
- Because approved methodologies for establishing legal traceability of length measurement for GPS do not currently exist under the Australian National Measurement Act (1960), GPS should not be used as the sole method of measuring length in legal surveys within Australia. Surveyors using GPS for legal purposes within Australia must adhere to the requirements of the appropriate verifying authority in the State or Territory.

2.6.3 Geodetic Datums and Geoid Separations

- The geodetic datum used in Australia is GDA (Geocentric Datum of Australia 1994). WGS84 is closely aligned with the International Terrestrial Reference Frame (ITRF), which in 2005, differs from GDA94 by about $^{3}/_{4}$ metre, due to plate tectonic movement. For most practical purposes GDA94 may be considered equivalent to WGS84, but if a WGS84 absolute accuracy of better than ½ metre is really required (i.e. not just relative accuracy), the current ITRF should be adopted. See the Glossary and the GDA Technical Manual for further information.
- The geodetic datum in use in New Zealand is NZGD2000 a geocentric datum in terms of the ITRF96 at epoch 2000.0.
- The WGS84 (World Geodetic System 1984) is the geocentric datum, used for broadcast and precise ephemerides associated with GPS satellite systems. See Glossary for further information.
- All adjustments of GPS data should be 3 dimensional on the GRS80 ellipsoid. For all practical purposes, the WGS84 ellipsoid is identical to the GRS80 ellipsoid.
- Horizontal survey measurements, once completed, should form a closed figure, and where possible, be connected to a minimum of two existing stations in the geodetic network with a Class appropriate to the survey being undertaken. Reference station coordinate values in the national geodetic datum should be obtained from the principal government survey organisation in the Australian State/Territory or New Zealand.
- The vertical datum used in Australia is Australian Height Datum (AHD).
- Where orthometric heights are to be calculated from the GPS observations, the selected control stations should have, when possible, accurate vertical datum heights. Otherwise additional GPS connections should be observed to BenchMarks with good vertical datum heights. These connections, along with the geoid model, enable fitting to the vertical datum
- Australian geoid separation values (N values) should be obtained from the latest AUSGeoid model, available from the Geoscience Australia Web site.
- Recent studies have confirmed that the geoid and AHD are not coincident over Australia, with a north south trend of about a metre. Using AUSGeoid98 differentially will eliminate this problem, but future versions of AUSGeoid will include the AHD-geoid difference so that it will produce AHD values directly.
- For more information please refer to the GDA Technical Manual at the ICSM Web site.

2.6.4 Equipment Validation

- If required, the equipment and software can be validated over existing, high quality geodetic network marks. The relevant authority can be contacted for more information.
- Another useful method is to measure a ‘zero baseline’, which is achieved by connecting a single GPS antenna to two GPS receivers using a special antenna cable splitter. The positions obtained from the two receivers should agree at the sub centimetre level.

2.6.5 Fundamental GPS Techniques

There are three fundamental GPS techniques:

- absolute point positioning (referred to in Part A, Section 4 and described in more detail in Part B section 2.6.14)
- differential GPS (using pseudo range measurement - DGPS)
- relative positioning (using carrier phase observations)

These guidelines generally refer only to **relative** GPS positioning, which requires two or more GPS receivers, observing carrier phase observations. The exception to this is the section on GPS observations for global/regional processing (Section 2.6.14) where only one survey quality GPS receiver is required, but the data collected is later processed with data from global and regional GPS sites, using on-line processing services.
It is the responsibility of the user to assess which GPS technique or combination of GPS techniques should be used to achieve the task being undertaken, having regard to the manufacturer's specifications for the equipment and survey specifications.

2.6.6 Planning a GPS Survey

2.6.6.1 Network Design and Geometry

- When planning a GPS survey, the first step should be choosing the appropriate technique for the precision required. Table 25 provides a guide to the user as to what technique should be used in order to achieve a particular class of survey.
- The location and distribution of points in a GPS survey do not depend significantly on factors such as network shape or intervisibility, but rather on an optimum layout with sufficient redundancy for carrying out the intent of the survey.
- All GPS surveys should be connected to the state control where it is available, for the purposes of survey integration, legal traceability and quality assurance. Such connection may be a regulatory requirement in some local authorities.
- The planning of the observations should be such that the error budget is sufficiently minimised. Redundancy in the observations is the best way of dealing with most of the error sources.
- Important issues are positive mark identification, centring, height of antenna phase centre as well as antenna orientation and independent reoccupation of the same point, after a sufficient lapse of time.
- Horizontal survey measurements, once completed, should form a closed figure, reference station coordinate values in the national geodetic datum should be obtained from the principal government survey organisation in the Australian State/Territory or New Zealand.
- A supplement or alternative to independent reoccupations could be the inclusion of conventional observations of appropriate accuracy (for example to create ties between unavoidably unclosed GPS polygons in the same adjustment).

Table 25 GPS Method vs Class

CLASS (Australia) **c-values for the CLASS(part A, 2.2.1)**	**3A** **≤1**	**2A** **≤3**	**A** **≤7.5**	**B** **≤15**	**C** **≤30**	**D** **≤50**
CLASS (New Zealand) (See the New Zealand Web site for details)	B10	M1	M10	M100		
Technique						
Classic Static	✓	✓	✓	✓	✓	✓
Quick Static			✓	✓	✓	✓
Stop and Go			✓ (1)	✓	✓	✓
Real Time Kinematic (RTK)			✓ (1)(2)	✓	✓	✓
Guide to minimum station spacing km (3)	5	1.5	0.5	0.1	N/A	N/A
Typical station spacing in km (4)	100-500	10-100	0.5-10	0.1-5	>0.05	N/A

Independent occupations per station (5) at least3 X (% of total stations)(6) at least 2 X (% of total stations)(6)	50% 100%	40% 100%	20% 100%	10% 100%		
Minimum independent baselines at each stn	3	3	2	2	2	2

Notes on Table 26:
N/A: Not applicable.
1. The Stop and Go method can achieve Class A with careful attention to the network design.
2. As the minimally constrained adjustment is usually not applicable in RTK, Class for this method is determined differently (see section "Analysis using misclosure comparisons". Class A and Order 1 can be achieved with careful attention to the network design.
3. Minimum station spacing is illustrated using a 5 mm noise level after adjustment. Below these minimum distances, special efforts are required to reduce the error budget. For a noise level of 10 mm these values are to be approximately doubled.
4. These values relate to the using of conventional equipment and proprietary software.
5. Independent occupations per station may be back to back, but the antenna should be re-set for each occupation. The minimum observation period should be observed with each occupation as per the manufacturers' recommendations.
6. For example for a CLASS A network aim for:
   (i) 20% of stations are to be occupied at least three times;
   (ii) 100% of stations are to be occupied at least twice.

2.6.6.2 Independent Baselines

- An independent baseline measurement in an observing session is achieved when the data used are not simply different combinations of the same data used in computation of other baseline vectors observed in that session.
- In the one session, observing with n receivers, the total number of baselines that can be computed is n(n-1)/2. However, only n-1 of those baselines are independent. The remainder – trivial baselines - are formed from combinations of phase data used to compute the independent baselines. The results from observations of the same baseline made in two different sessions are independent.
- Generally independent baseline processors assume that there is no correlation between independent vectors. Trivial baselines may be included in the adjustment to make up for such a deficient statistical model. If the mathematical correlation between two or more simultaneously observed vectors in a session is not carried in the variance-covariance matrix, the trivial baselines take on a bracing function simulating the effect of the proper correlation statistics, but at the same time introducing a false redundancy in the count of the degrees of freedom. In this case the number of trivial baselines in an adjustment is to be subtracted from the number of redundancies before the variance factor (variance of unit weight) is calculated. If this approach is not followed, trivial baselines are to be excluded from the network altogether.

2.6.7 General Requirements for GPS observations

The following guidelines refer to different types of GPS survey techniques such as static, quick static, pseudo-kinematic, post processed kinematic (stop/go) and real time kinematic (RTK).

- Users should be familiar with the procedures and recommendations contained in the GPS equipment and software manuals and the National and State/Territory survey standards and specification documents.
- In the event of a conflict between these Guidelines and the manufacturer's instructions, the manufacturer's recommendations will prevail. As part of the process of keeping these guidelines

current, any conflict between these guidelines and the manufacturer's recommendations should be forwarded to the ICSM Geodesy Technical Sub Committee representative with all relevant details to allow the conflict to be resolved.

- All ancillary equipment must be in good adjustment and repair and operated competently by trained personnel. This is of particular importance with GPS because it is a three-dimensional (3D) technique requiring accurate location of the antenna horizontally and vertically over any survey mark. All GPS measurements relate to the exact location of the antenna. Therefore its precise relationship to the ground mark is critical to the process of obtaining quality results.
- Receivers and baseline reduction software are to be of the "geodetic" type.
- Only carrier beat phase observations using two or more receivers for baseline measurements are considered in these guidelines (noting that Quick Static techniques are advantaged in the calculations by access to pseudo range observations).
- Satellite geometry during the field observation phase of any survey must be sufficient to ensure accurate results. The maximum geometric dilution of precision (GDOP) should be no greater than 8. The user should comply with the GPS manufacturer's recommendation's on GDOP values during observation periods. This can aid in resolution of integer ambiguities if required when using the GPS manufacturers processing software.
- The elevation mask should be generally set according to the manufacturer's recommendations, but typically should not be less than 15°.
- Inaccurate starting coordinates adversely affect the accuracy of the baseline results. Therefore, an initial geocentric coordinate within 10m of the true position should be used for the reduction of observations. With the cessation of Selected Availability and the improvement in the receiver algorithms, the receiver-generated position is usually within this limit. Use GDA or WGS84 coordinates in Australia and NZGD2000 or WGS84 coordinates in New Zealand.
- Low accuracy survey applications will not be affected by the quality of starting coordinates.
- It is not necessary to take meteorological readings. Use the GPS reduction software defaults for tropospheric modelling.
- The post processed and real time kinematic (RTK) techniques typically involve radiation from a base station and least squares network adjustment may not be appropriate. Notwithstanding this, some RTK receivers may allow users to download GPS baseline components (rather than coordinates) suitable for input to a least squares adjustment (particularly when independent check measurements (EDM etc) are added.
- Antenna heights for re-occupations are to be changed by at least 0.1 m unless set up on a pillar.
- The GPS signal may be degraded or blocked by nearby buildings, trees or topography. There should be clear visibility to the sky in all directions, down to the elevation mask being used (typically 15°)
- Multipath can be a significant source of errors, particularly when short occupation times are used. A typical high multipath environment is in the proximity of corrugated iron roofs, wet trees, high rise buildings and chain wire fences. As well as its direct effect, multipath appears as noise and can affect ambiguity resolution. Where multipath is likely, occupation time should be increased to allow the effect to be averaged away as satellite geometry changes.

2.6.8 Specific observational requirements for various relative GPS techniques

Within the fundamental relative GPS positioning technique there are several methods that have developed since the introduction of GPS. They all employ carrier phase measurements. Whilst most of the observational requirements are comparable, there are also some specific conditions:

2.6.8.1 Classic Static Baselines

The following guidelines apply to Classic Static baselines, in conjunction with the general guidelines:

- The observation period for shorter lines (approximately 10km) should be at least 30 minutes, except for specified applications in New Zealand where the minimum observation period can be 15 minutes. Observation periods for longer lines should increase as stated in the manufacturer's specifications or in accordance with any National or State/Territory specifications.
- The epoch recording rate is recommended to be 15 or 30 seconds.
- The satellite geometry should change significantly during the observation period.
- At least four, but preferably as many satellites as possible should be common to all survey sites simultaneously occupied
- Dual frequency receivers are preferred but single frequency survey quality receivers may be used for short lines (less than 10km.) for non high precision applications.
- Sufficient data should be collected to resolve ambiguities. This is particularly important for lines less than 15km.

2.6.8.2 Quick Static Baselines (Also known as Rapid Static or Fast Static)

The following guidelines apply to Quick Static baselines, in conjunction with the general guidelines:

- Enough data should be collected to resolve ambiguities. Please refer to the manufacturer’s recommendations in relation to the length of observation periods, number and geometry of satellites and the suitability of single or dual frequency receivers.
- Multipath can be a significant source of errors, particularly when short occupational times are used and special attention should be paid to this issue.
- The epoch recording rate normally may vary between 5 and 15 seconds.

2.6.8.3 Post Processed Kinematic Baselines (Also known as Intermittent or Stop/Go)

The following guidelines apply to Kinematic baselines, in conjunction with the general guidelines:

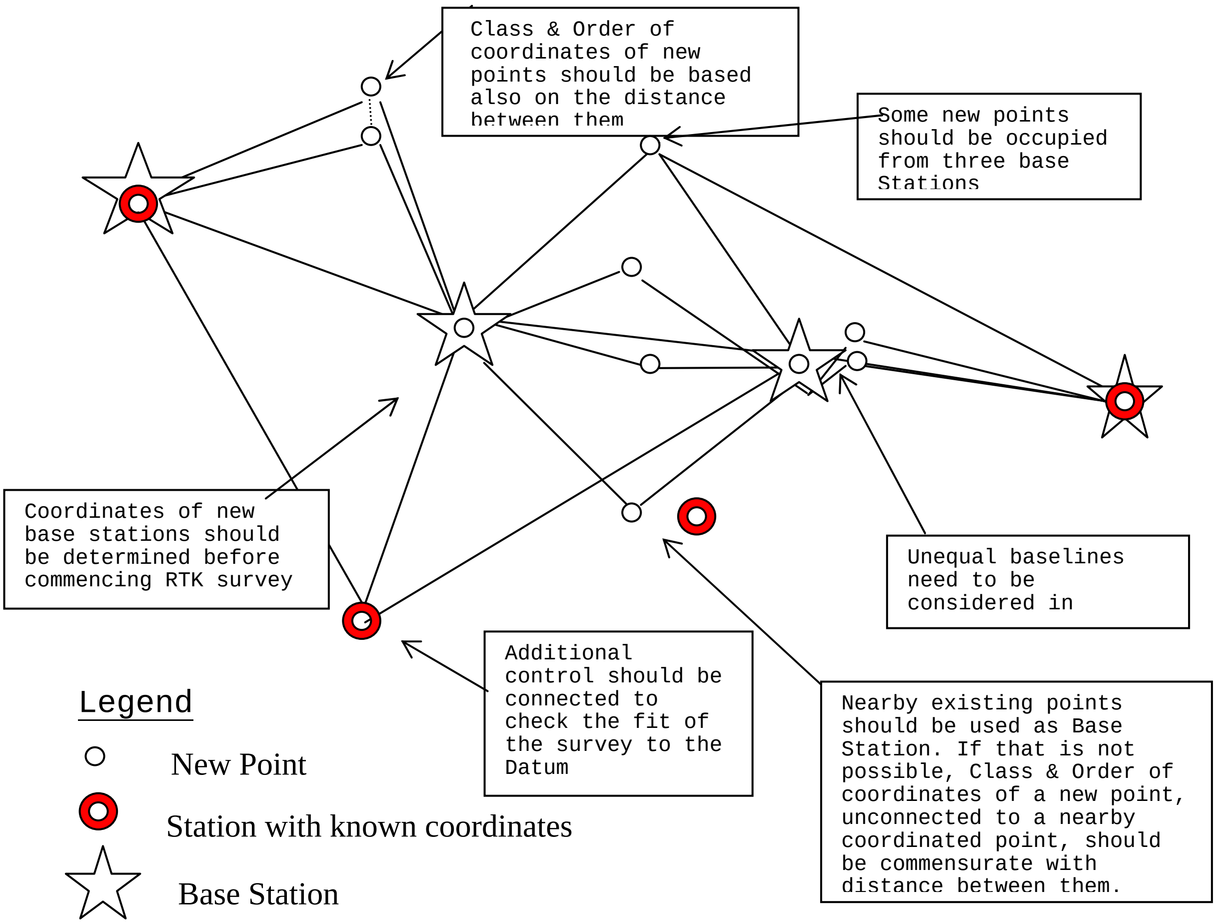
- Preferably five or more common satellites are required due to the likelihood of signal loss during motion between rover stations.
- Receivers should be initialised as per the manufacturer's instructions at the start of each kinematic chain so as to ensure ambiguity resolution has been achieved. (This is not necessary with receivers with ambiguity-resolution-on-the-fly capability.) The chain should close off on a known point. For completely independent results and for quality control purposes, each point should be re-occupied in a different session with different satellite geometry.
- The epoch recording rate should normally be between 1 and 5 seconds, but can be up to 15 seconds.
- Minimum station occupation should be between 5 to 10 epochs.
- Multipath can be a significant source of errors, particularly when short occupational times are used and special attention should be paid to this issue.
- Single frequency geodetic quality receivers may be used, although dual frequency capability is an advantage for cycle slip repair during processing.

2.6.8.4 Real Time Kinematic (RTK)

The Real Time Kinematic (RTK) GPS Surveying technique involves data from a fixed *base* receiver being sent by radio telemetry to a rover receiver and processed in real time to produce three dimensional coordinates of the rover. The following guidelines apply to RTK surveys, in conjunction with the general guidelines:

- Single frequency geodetic quality receivers may be used, although dual frequency capability is an advantage for ambiguity resolution and mitigation of the effects of ionospheric delay on longer baselines.
- The typical range for RTK is up to 15km although some manufacturers state that their equipment functions over longer ranges (e.g. up to 40km). Meeting accuracy criteria may limit the range to around 10 km (see also section 2.6.14).
- Precision claimed by most manufacturers is 10mm plus 2ppm or better (1 sigma, horizontal, at 1Hz). It should be noted that some manufacturers claim different precision for horizontal versus vertical and state that the precision varies according to the real time update rate.
- Real time update rate may vary according to the application, eg 1 update per second (1 Hz) through to 5 per second (5 Hz).
- Ambiguities must be resolved for all occupations. Sufficient data should be gathered to ensure ambiguities are resolved and users should adhere to manufacturer’s recommendations regarding minimum number and geometry of satellites and maximum PDOP.
- Multipath can be a significant source of errors when short occupation times are used. Special attention should be paid to this issue. Both, base and rover receivers should be located in a low multipath environment. Where multipath is likely at a rover site, occupation time should be increased to allow the effect to be averaged away as satellite geometry changes.
- To allow sufficient change to the satellite constellation being used and improve detection of errors such as multipath, re-occupations should be made more than 45 minutes apart and with an independent ambiguity resolution.
- Two independent occupations of all new stations from two base stations are a recommended minimum. Such re-occupations are the most reliable means of checking against systematic or gross errors. It is desirable that some of the new stations in each RTK survey are re-occupied from a third base station or checked using conventional observations.
- Typically both or all base-stations should have known three-dimensional coordinates. Use of at least two known base stations checks that no anomalies occurred at either of the base stations or at any of the new stations and that the new survey is consistent with the datum.
- Where a base station is one of the new stations in the survey (eg due to it having better radio coverage than a known station or where the existing control is scarce), it is prudent to occupy another known station with the rover. Values from this occupation should be also used to derive a mean value of the new station before it is used as a base station.
- New base Stations on very large projects should be surveyed using static or fast static GPS methods and coordinates should be calculated before commencing RTK.
- Sufficient additional control points should be occupied (as base and/or rover stations) to ensure redundant connection and fitting to the horizontal and vertical datum in the project area.
- The following quality attributes should be ideally logged with the derived coordinates: Base Station identification, Date, Time, Datum, Number of satellites observed and standard deviations of the derived coordinates.

Figure 2 Typical RTK Applications



2.6.9 Processing Baselines

In relation to the above-described techniques, the guidelines for processing GPS baselines are as follows:

- Surveys requiring a higher Class than 3A are not covered by this document and specialist advice should be sought. Such surveys would typically involve advanced techniques involving a multistation processor and precise ephemerides, as opposed to a baseline processor and broadcast ephemerides.
- The quality of the results of a GPS survey is determined by both the method of observation, including choice of equipment, and the quality of the reduction, adjustment and transformation procedures. The initial satellite datum station position for any baseline calculation should be in error by no more than 10 metres for each part per million accuracy required and is best obtained by a transformation, or by connection to another point with its coordinates known in the satellite datum.
- Because of the effect of the ionosphere, dual frequency receivers are used on lines over a certain length. "L1-only" solutions often show less noise for vector lengths below 10-15 km. Single frequency receivers can still satisfy Class A, B, C, D etc. requirements up to 20-odd km, but need an increasing number of hours of observation if the higher Classes of survey or longer baselines are observed. Dual frequency ambiguity fixed L1 and L2 solutions in their ion free linear combination are usually obtained for vector lengths above 10-15 km to up to 40 or 50 km. An ambiguity fixed solution is preferred, but the longer the distance becomes the harder it is to achieve this. Ion free ambiguity float L1/L2 solutions become more common for vectors of over 40 or 50 km in lengths up to about 90 km.
- For longer baselines eventually even triple difference solutions are used, if the observation duration is sufficiently long, to enable a sufficient change in the satellite geometry during the recording session. As a guide use 30 minutes + 20 minutes per 10 km of the baseline length.

- The reduction procedures outlined in Table 26 give a broad overview of the essential components that should be accounted for when undertaking the reduction of GPS data. Adhering to the procedures in this Table does not remove the necessity for statistical analysis of the results. The tabulated format has been used so that the reader can obtain a clear picture of the specific reduction requirements for achieving a given geometric **CLASS** of survey.
- The reduction procedures in Table 27 indicate recommended **minimal** requirements.

Table 26 RTK Recommended Processing Requirements

CLASS (Australia) c-values (one sigma)	**3A ≤1**	**2A ≤3**	**A ≤7.5**	**B ≤15**	**C ≤30**	**D ≤50**	**E ≤100**
CLASS (New Zealand) (see the New Zealand Web site for details)	B10	M1	M10	M100			
Baseline length	Recommended processing requirements						
<8 km	D*, DD, FX	D*, DD, FX	S, DD, FX	S, DD, FX	S, DD, FX	S, DD, FT	S, DD, FT
8-25 km	D, DD, FX	D, DD, FX	D, DD, FX	D, DD, FX	S, DD, FX	S, DD, FT	S, DD, FT
25-50 km	D, DD, FX(25)-FT(50)	D, DD, FX(25)-T(50)	D, DD, FX-FT	D, DD, FX-FT	D, DD, FX-FT	D, T	D, T
50-90 km	D, DD,FT	DD or T**, D, FT	DD or T**, D, FT,	DD or T**, D, FT,	DD or T**, D, FT	D, T, NCP	D, T, NCP
>90 km	D, T	D, T	D, T	D, T	D, T	D, T, NCP	D, T, NCP

Notes on Table 27:

S = single frequency D= dual frequency
DD = double differences FX= ambiguity fixed solution
FT = ambiguity float solution, with repaired cycle slips
T = triple difference solution with sufficient observation length, allowing change of geometry.
NCP= Narrow correlation, C/A code or Pseudorange methods, e.g. DGPS
* = L1 solution, from a dual frequency receiver, in order to enable ambiguity resolution by widelaning.
**= Double difference preferred, triple difference solution increasingly acceptable the longer the distance, if the observation length allows sufficient geometry change.

2.6.10 Analysis Using Least Squares Adjustment

In the case of classic static and quick static, least squares adjustments of the network, both minimally constrained and constrained by all suitable geodetic stations coordinates, should be carried out to verify that the survey meets the required standards.
Most proprietary baseline processing packages contain a suitable least square adjustment module. However, separate, specialised 3-D adjustment software should be used when adjusting a very large number of baselines, when combining GPS and terrestrial observations or when the proprietary software does not produce relevant statistical output.

2.6.10.1 Unconstrained Adjustment

- The processing software should be able to produce the variance/covariance statistics of the observed Cartesian vectors so that these can be input to a three-dimensional adjustment program. A least squares adjustment should be performed when deriving values for control surveys. This software should be capable of determining transformation parameters between the observed Cartesian vectors and the local geodetic system (Refer to Part B, Section 5 and 6.)

- Error ellipses should be calculated, after a minimally constrained least squares adjustment. They mainly prove quality of the net design rather than the quality of the observations. The error ellipses should be scaled by the apriori variance of unit weight (generally equal to one), unless the a-posteriori estimate of variance does not pass the Chi-square test. In case of the latter, the observations, the statistical model or even the mathematical model should be examined, the problem remedied and the adjustment rerun. In the case of not being able to remedy the situation, the error ellipses should be scaled by the a-posteriori variance factor.
- To confirm the quality of the observations, the standardised residuals should be checked for outliers, and these should be dealt with. The checking of the statistics often involves critical evaluation of the apriori standard deviations of the observations. If the baseline variance/covariance matrix is routinely modified by a multiplier, documentation of a measurement over a test network can be required as confirmation of the multiplier used.
- In order to conform to the internal consistency requirements for a particular geometric accuracy Class the following conditions should be met: the error ellipses should confirm the capability of the network design to meet the specifications, the standardised residuals and the estimate of variance should confirm that the observations have actually met the required standard (Refer also to Part A, Section 2.2.1).
- All points in a survey should conform to specifications belonging to the relevant classification. This is irrespective of whether the points are connected by baseline observations or not. This is also valid when relative accuracy values are calculated to points with previously established coordinate values.
- Geoid separation values are now applied to orthometric heights of points that will be constrained in the transformation and adjustment.

2.6.10.2 Application of Geoid Separation Values

Australian geoid separation values should be obtained from the latest AUSGeoid model, available from the Geoscience Australia Web site.

2.6.10.3 Constrained Adjustment

The final step is a fully constrained Least Squares adjustment. This adjustment is subjected to the same analysis as the above minimally constrained adjustment. Again error ellipses are calculated and the network is allocated an accuracy Order which enables its orderly integration with the database containing the existing data set of established coordinates (Refer also to Part A, 2.2.2 and Part B, Sections 5 and 6).

2.6.11 Analysis Using Misclosure Comparisons

For some GPS observation techniques, “new stations” are coordinated by radiation from “base stations”. A prime example of the approach is Real Time Kinematic (RTK) but it could also be true of Quick Static using only two receivers. With such techniques, there may not be direct measurements between the new stations. Least squares adjustment may not be appropriate for such techniques and analysis using misclosure comparisons may be sufficient. No matter what Order is required for the final coordinates, a minimum of two independent occupations of all new stations in the survey should be made using two base stations and the resultant 2D coordinates compared.

2.6.11.1 An Example of Misclosure analysis

The use of misclosure analysis, rather than least squares, is best explained using an example. A possible survey is shown in the Figure 3 below. The Client requires new stations (A to E) to be coordinated along a road corridor. The Client requires a relative horizontal accuracy between the new stations of 0.050m (95% confidence). The client may alternatively require that coordinates of all new points be of (say) Order 2. All new stations are occupied using a base station at Base 1 and then all are reoccupied using Base 2. For example, for the new station B, the occupation using Base 1 produces a set of coordinates for B at B_1 and the occupation using Base 2 produces a set of coordinates at B_2.

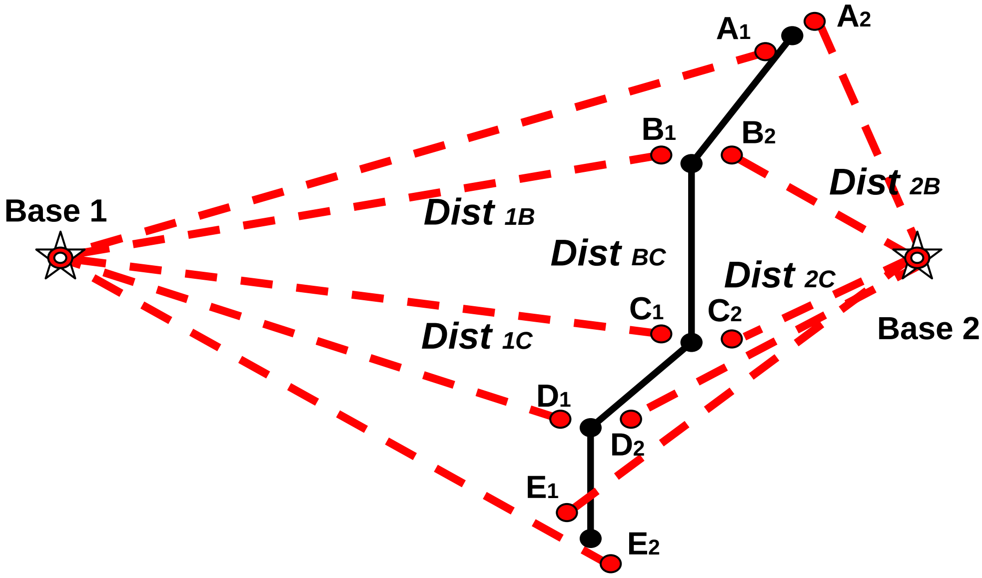


Figure 3 RTK Analysis

2.6.11.2 Testing the Survey

To test the survey, it is necessary to consider the misclose between the occupations at each new station (e.g. between B1 and B2). The standard deviation of the misclose (eg SD $_{\text{MISCL B}}$) can be calculated according to the error in the observed GPS baselines. An estimate of that error can be based on the manufacturer's specification for the GPS observation technique (e.g. according to the GPS baseline lengths and expressed as X metres + Y parts per million). Then, if the standard deviation for the GPS Baseline from Base 1 to B_1 is SD_{1B} and the standard deviation for the Baseline from Base 2 to B_2 is SD_{2B} , then the standard deviation of the expected 2 dimensional (horizontal) misclose SD_{MisclB} can be calculated as:

$$SD_{Miscl.B} = \sqrt{SD_{1B}^{\ \ 2} + SD_{2B}^{\ \ 2}}$$

If the actual 2 dimensional misclose vector between B_1 is B_2 is less than the standard deviation of the $SD_{Miscl\ B}$ * 2.45, then the GPS observations are agreeing within the manufacturer's specifications at the 95% confidence level.

Once the GPS observations have passed this misclosure test, the coordinates of station B can be calculated as a mean of the coordinates of B_1 and B_2. Often a simple arithmetic mean will suffice. However, if the two baseline lengths are significantly different, then a weighted mean may be more appropriate.

Using a simple approach, that treats the coordinates of the Base stations as error free, the standard deviation of the mean coordinates of B can be calculated as follows:

$$SD_B = \sqrt{(SD_{1B}{}^2 + SD_{2B}{}^2)/2^2}$$

In a similar manner, observations to all the other new stations could be misclose tested and standard deviations of the resulting mean coordinates could be calculated.

To test whether the survey is meeting the client's requirements, the standard deviation of a vector between the mean coordinates of new stations B and C can be calculated as follows:

$$SD_{BC} = \sqrt{SD_B{}^2 + SD_C{}^2}$$

To meet requirements (at 95% confidence), SD_{BC} * 2.45 must be less than the 0.050m specified by the client.

It should be noted that consistently large misclosures may indicate poor coordinates at one or both existing marks used as a Base or that these marks could have been disturbed. An individual large misclose could indicate a poor GPS solution.

2.6.11.3 A Worked Example

The above can be further illustrated by considering an example survey with the following characteristics:

- The observation technique is RTK with a 1 second update rate (1 Hz) for which the manufacturer's specification leads to an expected standard deviation of 0.010m + 2 ppm.
- The distance from Base 1 to new station B ($Dist_{1B}$) is 5km
- For convenience, let the distance from Base 1 to new station C ($Dist_{1C}$) also be 5km
- The distance from Base 2 to new station B is ($Dist_{2B}$) 1km
- For convenience, let the distance from Base 2 to new station C also be ($Dist_{2C}$) 1km

Using these characteristics and the procedure and formulae outlined above, the following analysis applies:

SD_{1B} = 0.020m (0.01m + 2 ppm over 5km)
SD_{1C} = 0.020m (0.01m + 2 ppm over 5km)
SD_{2B} = 0.012m (0.01m + 2 ppm over 1km)
SD_{2C} = 0.012m (0.01m + 2 ppm over 1km)

The combination of SD_{1B} and SD_{2B} gives the standard deviation of the misclose at station B as follows: $SD_{\mathrm{MisclB}} = \sqrt{0.020^2 + 0.012^2} = 0.023\mathrm{m}$.

Then, if the actual 2 dimensional misclose vector between the 2 GPS occupations at B (B_1 and B_2) is less than 0.056m (2.45 * 0.023) then the observations are meeting the manufacturer's specifications (at 95% confidence).

The standard deviation of the mean coordinates of B, SD_B = 0.012m.
Similarly, SD_C = 0.012m.

SD_{BC} can then be calculated as SD_{BC} = 0.017m $(\sqrt{0.012^2 + 0.012^2})$.

Then the 95% confidence standard deviation between B and C is 0.042m

(0.017 * 2.45), which is less than the 0.050m specified and a survey with these particular design characteristics does meet the client's requirements.

2.6.11.4 Improving the Survey Design

There are several options for improving the survey design, if the results in the above example do not meet the client's requirements:

(i) Use a more precise technique for the longer baselines. This would decrease the size of SD_{1B} and SD_{1C} in the above worked example.

(ii) Replace Base 1 with a new base station closer to the project area. This would also decrease the size of SD_{1B} and SD_{1C} in the above worked example.

(iii) Add a redundant observation from a third base station. Depending on the length of the baseline from Base 3 to (eg) station B, this could significantly improve the standard deviation of the mean coordinates of B: $(SD_B = \sqrt{(SD_{1B}{}^2 + SD_{2B}{}^2 + SD_{3B}{}^2)/3^2})$ and therefore decrease the size of the $SD_{BC.}$

(iv) Add another independent occupation from one of the base stations, typically the closer one (Base 2 in this case). This would have a similar effect as in (iii) above.

(v) Add more redundancy by measuring directly between the new stations. In such a case it may be more appropriate to use least squares adjustment, rather than this simplified closure analysis approach.

The appropriate option to take will vary from project to project according to logistical factors.
In the example survey outlined above, *option iv* may be the most effective. However, if the survey was not so linear and the new stations were spread more evenly across the project area, many new stations may not be meeting requirements due to the distances to one Base station being too long. In such cases, it may be more efficient to establish a higher density of base stations (*option ii*), rather than increasing the number of occupations (such as in *option iii or iv*).

It should also be said that where direct measurements are required to increase redundancy (*option v*), it may be more appropriate to move away from RTK and observe a networked approach using quick static.

2.6.11.5 Assigning Class and Order

Returning to the RTK example, once the observations are meeting the requirements of the survey, it is possible to test and assign Class and Order. Meeting specifications of a given Class and Order may even be the client's requirement. The following points should be noted:

- Class and Order should be assigned based on the relationship between new stations as well as between new and existing stations, even though there may not be direct observations between them.
- With an RTK survey such as this, Class and Order are effectively assessed in the same way. This is because the coordinates of the new stations result from the Base stations being held fixed in computing those new coordinates.
- In this example, the Class between stations B and C and the Order of B and C can be tested by comparing SD_{BC} to the required "r" value, based on the distance between stations B and C (using the same formula as in sections 2.2.1 & 2.2.2 in Part A).
    - For example, if the distance between B and C is 1km, then Class B and Order 2 (say) require an "r" value of 18mm (that is 15 * (1 + 0.2)). Therefore, if SD_{BC} is less than 44mm (2.45*0.018m), then the survey satisfies Class B between stations B and C and therefore Order 2 for these coordinates.
    - For short distance B-C and in general, SD_{BC} of 20mm is considered to be satisfactory for

Class, A, B, C and for Order 1, 2, 3, regardless of what "r" value is derived from the above formula.

- Order should be assigned according to the Class between the stations in question and all surrounding stations.
- It should also be remembered that the Order assigned to the new stations could never be higher than the Order of the base stations. Therefore, the coordinates of stations B and C can only reach Order 2 if the coordinates of Base 1 and Base 2 are Order 2 or better.
- If there is a nearby existing control station and it is not used as a base station, it too should be considered in assigning Class & Order to the new stations. Ideally the existing control station should be occupied as though it is a new station. That will check homogeneity of the existing network and improve integration of the new survey into it.
- If the existing control station cannot be occupied, Class and Order for any nearby new stations should still be calculated as though there is a connection (similarly to assigning Class & Order to stations B and C above). The calculation should be based on the standard deviations of other observations from the new station and on approximate distance between it and the existing station.

2.6.11.6 Some Comments on Height

This above approach to analysis by misclosure can be applied to heights, subject to the following points:

- Some manufacturers recommend a specification for heights that is different to horizontal. In the absence of such specific recommendations, the standard deviation for ellipsoidal height difference from a GPS baseline should be 1.5 times that of the horizontal component.
- For calculating 95% confidence intervals for the single dimension of height, 1.96 should replace the 2.45 value used above.
- Misclosure analysis using comparison of heights from independent occupations is best done using ellipsoidal heights of the new stations and based on the ellipsoidal heights of the base stations being as consistent as possible. That is, error due to variation in the geoid or local vertical datum should be minimised.
- The client's requirements for height may need to be in terms of the local vertical datum and any testing and reporting of the final height results may need to account for any extra error in fitting the GPS ellipsoidal heights to the local vertical datum.

2.6.12 Field Notes and Data Lodgment

- Field observation recording sheets (log sheets) should be completed for each session. The receiver type, serial number and software used for reductions should be recorded on these sheets.
- An indication of independent checks on height of antenna is essential.
- Field Book (or log sheet) should contain a sketch of the relevant part of the network as well as the name and/or identifier (ID) used for each station. Each baseline measured should be clearly indicated. The reduction software may produce a printout of these details.
- The GPS field recording sheets should be made available should an examining authority so request.
- See Appendix A for an example of a log sheet, recommended for use on high accuracy and geodetic GPS surveys.

Note: These requirements may differ in each Australian State/Territory and New Zealand.

2.6.13 Digital Data Storage

- Raw observational data should be archived in case an auditing process is required by the examining authority. (Note: raw data is equivalent to the surveyor's field book and should be retained for the same length of time)
- If required by the examining authority, result files from the baseline processing and final adjustments MUST be supplied in digital form. The recipient may recommend the processing and/or adjustment software digital format. This enables automatic inclusion of the results in the recipient's data base systems.
- Final adjusted coordinates are to be provided in the coordinate system specified by the relevant National or State/Territory surveying organisation.

2.6.14 "Absolute" Positioning with GPS

Accurate positions can be determined by GPS observations using a single geodetic quality receiver. Although this relies on a regional or global framework of continuously operating geodetic GPS stations, it is generally transparent to the user. The data can be processed by a variety of methods, including the Australian AUSPos service (www.ga.gov.au/nmd/geodesy/sgc/wwwgps/), which automatically provides the results in terms of GDA94, or several overseas systems that provide the results in terms of the International Terrestrial Reference Framework (ITRF) (e.g. JPL's AutoGypsy and the Canadian CSRS-PPP). It is also possible to produce similar results from regional processing of geodetic GPS data using specialised software (e.g. Bernese, Gamit).

Table 27 gives guidelines to achieve typical Positional Uncertainty values using this type of technique. However, as this technique depends on many local & global variables the results may vary from the guidelines shown below. These guidelines do not override those which may be recommended by individual jurisdictions.

Table 27 GPS data attributes for "absolute" positioning.

Positional Uncertainty (m) [1] **(Horiz Vert)**	0.025 0.05	0.05 0.1	0.1 0.2
Location [2]	Australia	Australia	Australia
IGS products [3] **(minimum standard accepted)**	IGS Final (~14 day delay)	IGS Rapid (~2 Delay)	IGS Ultra-rapid (partly predicted)
GPS Receiver [4]	Geodetic, dual frequency, carrier phase & code	Geodetic, dual frequency, carrier phase & code	Geodetic, dual frequency, carrier phase & code
GPS Antenna [5]	IGS/NGS modelled	IGS/NGS modelled	IGS/NGS modelled
GPS data format [6]	RINEX	RINEX	RINEX
GPS data sampling [7]	30 sec	30 sec	30 sec
Duration of observations [8]	Multiple 24 hour sessions	Multiple 6 hour sessions	Multiple 2 hour sessions
Repeatability between sessions (m) [9]	0.025 0.05	0.05 0.1	0.1 0.2
Transformation to GDA94 [10]	Yes	Yes	Yes
Solution statistics satisfied [11]	Yes	Yes	Yes
Antenna type [12]	Make, model & serial number	Make, model & serial number	Make, model & serial number
Antenna height [13]	mm	mm	mm
Reference stations [14]	At least 3 within 1500 km	At least 3	At least 3

1. Positional Uncertainty is a 95% confidence value, in metres, with respect to the datum, which in Australia is GDA94 (see Part A, Section 4).
2. The processing systems will work anywhere in the world, but outside Australia, or with non-Australian processing systems, results are generally given in terms of the International Terrestrial Reference Frame (ITRF) at the epoch of the survey.
3. Refer to the IGS product guidelines at http://igscb.jpl.nasa.gov/components/prods.html to see the usual delay for the various IGS products to become available. Some services may use their own products equivalent to IGS's (e.g. orbits, earth orientation, satellite clock corrections). The use of the IGS ultra rapid products may sometimes produce results with unacceptable uncertainty.
4. Some hand-held receivers may provide phase & code, but the quality of their data cannot be guaranteed for this type of processing
5. The processing must account for antenna phase centre variation with the azimuth & elevation of satellites. See ftp://igscb.jpl.nasa.gov/igscb/station/general/igs_01.pcv & http://www.ngs.noaa.gov/ANTCAL/ for the latest list of antenna calibrations. The standard naming convention should be used to eliminate ambiguity, see ftp://igscb.jpl.nasa.gov/igscb/station/general/rcvr_ant.tab (see also note 12).
6. Most commercial geodetic GPS software packages will convert the proprietary observed data to the Receiver Independent EXchange format (RINEX) (see ftp://igscb.jpl.nasa.gov/igscb/data/format/rinex210.txt for a full explanation). TEQC is a freely available quality checking package that also converts the most popular geodetic GPS receiver data types to RINEX format, see www.unavco.org/facility/software/preprocessing/preprocessing.html for details.
7. Most processes use 30 second data, but will accept any sampling rate less than 30 seconds that can be stripped back to 30 seconds (e.g. 1, 3, 5, 6, 10, 15, 30 sec).
8. Each session should be entirely within a UT day. Although shorter duration sessions may give adequate results, they cannot be guaranteed, particularly if local conditions are unfavourable (e.g. multi-path, interference, obscured sky view). Repeat shorter duration sessions should be observed at different times of the UT day to minimise systematic effects from the GPS system and ambient site conditions (e.g. similar satellite constellation).
9. Multiple sessions are recommended to ensure repeatability and hence confidence in the result. If two sessions do not agree within the required precision, a third session is required to resolve the discrepancy. Equipment should be set up again at the commencement of each session, as per normal multi observation geodetic practice, to isolate setup errors.
10. Transformation to the local datum is required (GDA94 in Australia). In Australia this is automatically done by the AUSPos processing system. For other systems which provide ITRF results, the time-varying ITRF-GDA94 transformation parameters published by Geoscience Australia are recommended (www.ga.gov.au/nmd/geodesy/techrpts/index.htm). Other methods may be used provided they are based on at least three fiducial positions known in both systems (ITRF and local datum) and include tectonic motion and transformation from ITRF to local datum.
11. To ensure that the data used is of an acceptable standard, the service provider's solution statistics must be examined and acceptable. This may vary between systems, but typically should include: estimated coordinate precision and observation fits.
12. The calibration for an antenna can be different, even for the same brand with only slight variations in the model. Exact identification is essential to ensure that the correct calibration is applied (see note 5).
13. If the results are to be reduced to a fixed survey mark, the vertical height of the Antenna Reference Point (ARP) above this mark must be accurately measured, preferably by several independent means. Check the manufacturer's specifications in conjunction with the IGS document at ftp://igscb.jpl.nasa.gov/igscb/station/general/antenna.gra and the NGS

documentation at http://www.ngs.noaa.gov/ANTCAL/ to identify the ARP, as it can vary subtly even within one manufacturer.

14. Most processing services will automatically select the nearest three IGS stations as reference stations for the processing. While this can generally be relied on, for critical projects the operation of appropriate reference stations and reliability of their ITRF station coordinates (e.g. sites affected by recent earthquake movements or GPS antenna changes) should be checked before proceeding.

Recommended Reading

Various papers and publications exist on all aspects of GPS and can be found in the references at the end of this publication - e.g. Burkard et al (1983), DoD (1991), McElroy et al (1992), Malys et al (1997), National Geodetic Survey (1986), NAVTECH, Sickle (1996), Torge (1991), Trimble Navigation (1988,1991,1992), Wells (1986).

GPS Observation Log
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LOCAL – UTC DAY OF YEAR _______
Local Time offset UT + _______

Station Name __________________________________ **Station ID**______________________ **Date** __/___/_____
Location__________________________ **City**_________________ **Project**________________________________
Observing Monument Inscription and description_____________________________________

GPS equipment: **Type** **Model No** **Serial No.** **Receiver Operator** ____________________
Receiver ____________ ____________ _____________ **Agency** _____________________
Antenna ____________ ____________ _____________
Receiver Software and Version __________________________
Collection rate _____**secs** **Elevation Mask** ______**degs**
☐ **Logging confirmed**
☐ **Data downloaded** Date __ / __ / ____
☐ **Backups made** ☐ Zip Disks ☐ Floppy ☐ Tapes
Disk/Tape File Name (eg 123_DDDS.*)
1 _________ ______________________
2 _________ ______________________
3 _________ ______________________
File naming convention used:

Equipment used
☐ Tribrach
☐ Tripod
☐ 240v AC and power converter
☐ 12v car battery
☐ Internal battery pack
☐ External battery pack
☐ _________________________

Timing:	**Local Time**	**Local Date**	UTC Time	UTC Date	UTC Day
Actual Start Time	____________	____________	____________	____________	___________
Actual End Time	____________	____________	____________	____________	___________

Daily Session Number _________ ☐ Power Failure – started over with new Log

Receiver Solution (record near end of session):
UTC Time ________________________ Height (metres) ___________________
Latitude ________________________ Longitude ________________________

Site Photographs/Sketch/Notes

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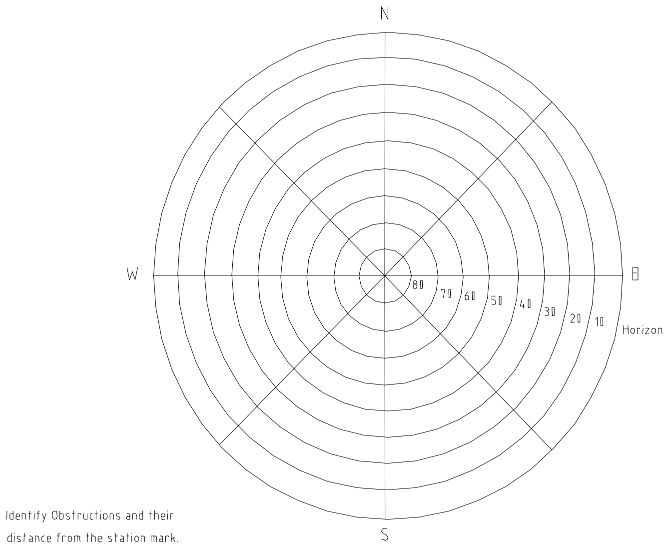

Antenna Sketch

Station Name ______________________ **Station ID**_____________________ **Date** __ / ___ / ______

Antenna Setup & Height information:

Include a sketch of the antenna setup above. Show all mounting accessories (tripod, pillar, tribrach) as well as heights measured. (Record antenna type and model number on the obverse page)

Record the measured height above the ground mark to as many defined positions on the antenna as possible.

Indicate (tick) whether the height measurements are slant or vertical.

For slant measurements, include the horizontal offset distances to the centre of the antenna.

Record the antenna vertical offset, ie height of Phase centre above the Rinex Antenna Reference Point (ARP).

It is preferred to enter in the receiver the ‘Rinex antenna height’. This is the vertical distance measurement, in metres, from mark to the Antenna Reference Point (ARP).

The ‘Rinex’ ARP varies from manufacture to manufacture and from model to model. The ARP is usually the bottom of the antenna pre-amp assembly. Check the manufacturer’s specification.

Check-measure all height measurements in feet/inches.

Height of Antenna:	**Vert. Dist**.	Slant Dist.	Horiz. Offset to Ant. centre
Mark to Top of ground plane	☐ ________ m	☐ _______ m	☐ _______ m
Mark to Bottom of ground plane	☐ ________ m	☐ _______ m	☐ _______ m
Mark to Bottom of choke ring	☐ ________ m	☐ _______ m	☐ _______ m
Mark to ARP	________ m	☐ **Height to ARP entered in receiver**	
Vertical offset ARP to L1/L2 Phase centre	________ m		
Mark to L1/L2 Antenna Phase centre	________ m	☐ True Vertical height entered in receiver	
		>>> **Do not enter slant distance in Receiver** <<<	
Check measures: feet/inches	________	_________ _________	_________
converted to metres	________	_________ _________	_________

Height Hook used YES ☐ NO ☐ Height Rod used YES ☐ NO ☐

Local Weather: ____

Notes on access, unusual features etc:____

____

____

____

____

____

2.7 INERTIAL SURVEY SYSTEMS (ISS)

2.7.1 Planning

2.7.1.1 Survey Design

ISS surveys may be traverses, or intersected traverses to form networks. Control points should be of an equivalent or higher order than that intended for the new survey. Refer to **Error! Reference source not found.** & Table 29 for survey planning and design considerations.

Table 28 Traverse Design

CLASS	**A**	**B**	**C**	**D**
Minimum number of connections to Control Points	4	4	3	3
Max distance between control points	*	30 km	60 km	60 km
Max travelling time between control points	*	1 hr	2 hr	2 hr
Traverse observed in forward & reverse direction	Yes	Yes	Yes	Yes
Control point at each major change of topography	Yes	Yes	Yes	Yes
Max distance between control points over topography with rapidly changing gravity gradients	*	10 km	20 km	20 km
Traverse corridor width not greater than one third of the straight line distance between Control points.	Yes	Yes	Yes	Yes

* As appropriate to the Survey.

Table 29 Network Design

CLASS	A	B	C	D
Minimum number of control points	8	8	8	8
A control point in each of the four corners of the network	Yes	Yes	Yes	Yes
At least one control point in centre of network	Yes	Yes	Yes	Yes

2.7.1.2 Calibration

Calibration of ISS equipment involves both:

- quality control of ISS surveys, and
- equipment calibration.

Quality control is achieved by monitoring system performance parameters which are estimated as a function of each survey.

Equipment calibration should be undertaken when performance parameters vary significantly from calibrated values.

Calibration should comprise:

- a static calibration performed annually or after maintenance,
- a dynamic calibration prior to inertial surveys.

The dynamic calibration should be performed between horizontal first order control.

2.7.2 Observation Requirements

Table 30 Observation Requirements

CLASS	**A**	**B**	**C**	**D**
Minimum duration of IMU alignment prior to commencement of traverse	60min	45min	45min	45min
Max period between alignments	6hrs	6hrs	6hrs	6hrs
Max period between Zero Velocity Updates in topography with ...				
• linear gravity profile	3.5min	3.5min	4min	4min
• rapidly changing gravity gradient	2min	2min	2min	2min
Accuracy of offset measurement system	*	$\pm$1cm	$\pm$1cm	$\pm$1cm
Accuracy of connection between Vehicle Reference Point & IMU	*	$\pm$1cm	$\pm$1cm	$\pm$1cm

* As appropriate to survey task.

2.7.3 Reduction Procedures

Traverse reduction should generally comprise:

- an onboard mission adjustment (OBMA) for field verification of survey precision;

- a post-mission adjustment (generally of a least squares type).

In general the differences between forward and reverse (or intersecting) traverses following OBMA, should satisfy the relevant accuracy specification for the **CLASS** of survey. (Refer to Part A, 2.2 Table 1)

2.8 HORIZONTAL CONTROL SURVEYS BY PHOTOGRAMMETRY

Table 31 Observation Requirements

CLASS	A	B	C	D
Targets	Yes	Yes	Yes	Yes
Pre-marked	Yes	Yes	Yes	Yes
Flying Height (m)	Max 4000	Max 4000	Optional	Optional
Photo-overlap: Forward Side	 66% 25%	 66% 25%	 66% 25%	 66% 25%
Comparator: Pointings per target not less than	4	2	1	
Pointings per reseau	4	2	Optional	Optional
Number of different reseau Intersections per target	4	2	Optional	Optional
Rejection limit from mean of target pointings (µm)	3	3	3	3
Bundle adjustment	Yes	Yes	Yes	Yes
RMS of adjusted photo-coordinates not more than (µm)	4	7	10	30
Camera calibration frequency (years)	2	2	2	2
Comparator calibration frequency (months)	6	6	6	6

3. STATION OCCUPATION

3.1 Proof of occupation should be recorded to enable conclusive and unambiguous identification of all observation positions occupied.

3.2 Optical plummets in good adjustment have a centring accuracy of 1.5mm, whilst a plumb-bob has an accuracy of 3.0 mm. Propagation of centring errors for instruments and targets should be modelled into weighting systems.

3.3 Eccentric stations should, if possible, be placed on line to distant control points. Three dimensional connections between control and eccentric stations should contain sufficient redundant measurements to enable independent fixation.

4. OPTIMISATION AND NETWORK DESIGN

4.1 GENERAL

It is necessary to confirm the ability of any geodetic network design to satisfy the required coordinate accuracy specifications.

Most least squares adjustment programs incorporate a "design" or "network optimisation" option to facilitate this process.

In general the following situations should be avoided:

- radiations, whether by EDM or GPS;
- short unmeasured connections between nearby points in a network (i.e. gaps shorter than 1/3 of the distances between surrounding points should be measured where possible).

4.2 OPTIMISATION

The optimisation procedure includes the generation of error statistics for all points and lines in a proposed geodetic network.

Formal error statistics are derived by manipulating elements of the inverted normal matrix. The elements of this matrix are defined by:-

1. The geometry of the network as described by the provisional coordinates and nominated observed lines.

2. A priori estimates of the precision of the observations.

It is important to note that the elements of the inverted normal matrix are in no way dependent on any actual observed quantities. It is therefore possible to generate error ellipse information for a proposed network, prior to field operations, by defining network geometry and expected observation precision.

Survey optimisation allows a network designer to experiment with different network configurations and different measurement precisions to ensure that the proposed network does satisfy requirements. Error ellipses for points and lines can be generated and compared with the accuracies required for these points and lines. Thus the density and accuracy of measurements can be decreased or increased until the optimal level is identified.

The following points should be noted :-

1. The accuracy estimates for intended observations must be realistic. The optimisation will only be valid if the a-priori standard deviations can actually be achieved during the course of field observations. Careful consideration must therefore be given to the impact of external error sources, e.g. horizontal refraction.

2. Network optimisation assumes the presence of normally distributed random errors only. It will not necessarily reflect sensitivity to observational blunders or systematic errors. Therefore experience of the network designer is required to ensure that the design conforms with good survey practice, e.g. sufficient redundancy.

Standard deviations for the various observations are listed in the appropriate sections of Part A.

4.3 CONNECTIONS TO EXISTING NETWORK

If surrounding control is introduced into the design, the influence of the stations selected for this purpose can be maintained by including them as constrained rather than fixed points. This can be effected by introducing them as position .

If these points were held fixed they would not contribute to the normal equations nor would they influence the simulation.

It should be noted that the use of position equations to constrain existing control will result in any correlation between these stations being ignored or, at best, approximated. To overcome this, the rigorous variance covariance matrix for these points resulting from the previous adjustment may be included in both the optimisation procedure and the final adjustment.

The principles outlined above apply to all one, two and three-dimensional networks.

5. NETWORK ADJUSTMENT ASSESSMENT

Network adjustment is necessary for assigning CLASS to a horizontal control survey and ORDER to the stations in that survey (Part A. 2.2). For survey networks that are to be incorporated into the national geodetic network, a one, two or three dimensional adjustment program commensurate with the accuracy of the survey, must be used.

5.1 ADJUSTMENT METHODOLOGY

5.1.1 Minimally Constrained Adjustment

A minimally constrained adjustment allows confirmation of the internal consistency of a network based on the precision assigned to each observation. The minimally constrained adjustment should be computed on the ellipsoid associated with the datum on which the observations were acquired. Where observations are on more than one datum it will be necessary to transform all sets of observations to the one common datum.

The results of the minimally constrained adjustment along with the field survey methods and reduction techniques should then be used to assign CLASS, as described in Part A, Section 2.2.1.

5.1.2 Constrained Adjustment

A constrained adjustment should be undertaken to fit the survey into the existing coordinate set. Where the observations to be constrained are on a different datum from the one required, they must be transformed as part of the adjustment process. In such a case, the ease with which the survey can be fitted to the existing coordinate set will be a function of the precision of the transformation technique as well as the precision of the observations and the homogeneity of the existing coordinate set. It is essential that all these factors be taken into account when performing the constrained adjustment.

5.1.3 One, Two and Three Dimensional Adjustment Programs

One-dimensional network adjustment programs are used exclusively for the adjustment of vertical control networks.

Classical two dimensional network adjustment programs have long been used for the adjustment of horizontal control surveys.

The advent of three dimensional measurement technologies (eg GPS) has led to three dimensional adjustment techniques where Horizontal and Vertical Control are difficult to separate. In such a case it is necessary to perform the adjustments with all available observations and to assign **CLASS** and **ORDER** separately for Horizontal and Vertical Control.

6. DATUM TRANSFORMATIONS

The latest information about transforming positions from one datum to another, and about transforming between ellipsoidal heights and the Australian Height Datum can be found in the Geocentric Datum of Australia Technical Manual.

P A R T C

RECOMMENDED MARKING PRACTICES

1. STATION MARKING PRACTICES

All ground stations pertaining to surveys that will be incorporated into the National Geodetic Data Base should be permanently marked. These include all first, second and third order horizontal control stations and bench marks along first, second, third and fourth order levelling routes. Spacing along levelling routes depends upon the CLASS of levelling and the purpose for which it is undertaken.

2. ESSENTIAL CHARACTERISTICS OF MARKING

Marks shall be made of corrosion resistant materials and placed in such a fashion that the long-term stability and safety of the marks are maximised. These qualities can be achieved by placing marks in stable ground or in solid rock, using good quality materials and robust construction techniques. In unstable areas, permanent marking is generally achieved by placing deep-seated marks that penetrate the surface soil to a depth that bypasses the zone of seasonal influence. To ensure unambiguous identification, the Station Identifier should be engraved or stamped on the mark, or durable tags with the identifier should be firmly attached.

2.1 RECOVERY MARKS

Preferably three recovery marks should be placed at first, second and third order horizontal control stations. They should consist of suitable corrosion resistant material set in situ such that their long term stability and safety are maximised.

3. DESIRABLE CHARACTERISTICS OF MARKS

3.1 SUB-SURFACE MARKS

Sub-surface marks may be established at stations considered to be in an unstable environment or at stations of particular importance. Sub-surface marks should consist of corrosion resistant material set in situ such that their long term stability is maximised. The sub-surface mark should be placed 100-125mm below the base of the surface mark and extreme care should be taken to ensure that the surface mark is centred directly over the sub-surface mark.

3.2 WITNESS MARKS

Horizontal and vertical control marks not otherwise identified by a beacon or similar structure should have witness posts placed nearby to assist in locating and protecting those marks. The exception would be where public safety would be endangered by so doing.

3.3 AZIMUTH MARKS

At permanently marked horizontal control stations where a reference azimuth cannot conveniently be obtained by sighting to an adjacent station, azimuths commensurate with the intended order of the station should be provided to suitable reference marks of a permanent nature.

3.4 LAND TENURE OF THE SITE

Where a choice is possible, marks should be placed away from private property and on Crown Land so that access problems in terms of right of entry are minimised.

P A R T D

RECOMMENDED DOCUMENTATION PRACTICES

and
DATA ARCHIVING POLICY
for a
NATIONAL GEODETIC DATA BASE

1. INTRODUCTION

The need for information systems for surveying and mapping has long been recognised. A survey is not finished until the computation and mathematical adjustment processes have been completed, the results assessed, and station information summarised, documented and made available

The public value of costly field surveys increases considerably if the resulting coordinates, and associated information, are readily accessible from databases containing standardised information of high integrity.
The following sections describe the desirable data elements to be held in these data bases, rather than a layout or design for paper documents.

2. HORIZONTAL CONTROL DATA ELEMENTS

It is desirable that data be presented to the user in a form that is indicative of the accuracy of data. Consequently, for surveys of CLASS D, geographical coordinates will normally be displayed to 3 decimal places of a second, while for survey of CLASS C and higher, geographical coordinates will normally be displayed to four decimal places of a second.

2.1 STATION IDENTIFIER

This is designed to provide a unique identifier for the station. A number of identifiers may be attributed to a station ranging from complete alpha or complete numeric, to a combination of alpha numeric. Usually no more than four identifiers are attributed to a station.

2.2 STATION ESTABLISHMENT TECHNIQUE

This is designed to give some information about the method or technology used to establish the coordinates of this station e.g. GPS, Traverse. This information gives subsequent users of the station some idea about possible clearing, Reference Objects etc.

2.3 OTHER OBSERVATIONS

The information included in this area is designed to increase the user's access to information of the type given in Station Establishment Technique, based on other observations made at the station.

2.4 RESPONSIBLE AUTHORITY

Indicates the authority responsible for the information. This information should indicate the State or Territory in which this authority operates.

2.5 DATE OF CURRENCY

This date is intended to reflect how up-to-date the information is. The currency date will be modified on the basis of the latest action on the data, whether it be in the form of reoccupation, additional observations, more recent adjustment etc.

2.6 LAST VISIT

This date is different to the currency date in that it refers only to the last field occupation of the monument, or to the last visit to the station.

2.7 DESCRIPTION OF STATION MARK

A full description of the monument is required. It should include information about:

- the stability of the mark;
- the type of mark, including its construction;
- its location relative to ground surface, structure etc.

2.8 BEACON

A beaconed station is of significant importance because of its potential use as an R.O. If the station is beaconed, a description of the beacon should be given e.g. steel quadripod with 4 vanes.

2.9 ACCESS

For the purposes of data base information, only limited information about access should be included. Three types of information are required.

1. The type of access, eg. 4WD, 30min climb or helicopter etc.

2. A reference to where more detailed information about access can be obtained.

3. Details of the owner and or occupier of the land on which the station is located.

Recognising that a station may be situated in an environmentally, historically or politically sensitive area, the record should highlight:

- whether or not the station is in a sensitive area;
- the nature of the sensitivity;
- a reference to the responsible authority.

2.10 PHOTO IDENTIFICATION

If the station has been identified on an aerial photograph, a reference to this information should be given. This reference should include:

- the authority that holds the photography;
- the year of the photography;
- the scale of the photography;
- any other relevant information;
- type (category) of identification.

2.11 HORIZONTAL DATUM

The horizontal datum being used needs to be adequately defined e.g. GDA, AGD, WGS84.

2.12 COORDINATES

Output will show both geographic and grid coordinates. The geographic coordinate set being used should be identified (e.g. GDA94, AGD84, WGS84). Station values should be input in the form of degrees, minutes, seconds and decimals (usually four) thereof.

2.13 HEIGHT AND DATUM

The height of the station, on the defined vertical datum, should be given in metres and decimals (up to three) thereof.
Preferably only “observed” heights should be stored, but if a “derived” height is stored it must be flagged as such and the link to the original “observed” quantity maintained (e.g. an ellipsoidal height may be “observed” with GPS and an AHD height “derived” by subtracting the best available geoid-ellipsoid separation, or, an “observed “ AHD height may be converted to a “derived” ellipsoidal height by adding the geoid-ellipsoid separation).

2.14 HORIZONTAL ADJUSTMENT

Identification of the horizontal network adjustment(s) used to define CLASS and/or ORDER should be given.

2.15 VERTICAL ADJUSTMENT

Identification of the vertical adjustment used to propagate height to the station should be given if appropriate.

2.16 RECOVERY MARK INFORMATION

Many control stations will have reference or recovery information associated with the primary monument. Details about these auxiliary monuments should include:

monument type and construction;

three dimensional coordinate information relative to primary monument including details of the form of this relative information (e.g. horizontal distance and height difference).

2.17 ADJOINING STATION INFORMATION

All stations observed from this station, either directly or indirectly, should be indicated. The method of observation should be included if appropriate.

2.18 CLASS

CLASS is a function of the precision of a survey network, reflecting the precision of observations as well as suitability of network design, survey methods, instruments and reduction techniques used in that survey. Preferably the CLASS is verified by an analysis of the minimally constrained least squares adjustment of the network. It is described by alpha character(s) e.g. A, PB etc.

2.19 ORDER

ORDER is a function of the CLASS of the survey, the conformity of the new survey data with an existing network coordinate set AND the precision of any transformation process required to convert results from one datum to another. It is described by numbers e.g. 1, 2, 3 etc. ORDER has been superseded by LOCAL UNCERTAINTY, but ORDER may continue to be used until LOCAL UNCERTAINTY is fully implemented.

2.20 FIELD RECORDS

An appropriate reference to the field records should be made, including the authority that holds these records.

2.21 ADDITIONAL INFORMATION

Certain technologies will have associated with them, information which to the expert user, will be of substantial use. This information will be defined in the recommended practices for the relevant technology and will include information about datums, transformation parameters etc.

Any other additional information that may be of assistance should also be added.

2.22 CADASTRAL INFORMATION

A reference to the property on which the monument is located, may be appropriate. However, this reference, will usually be obtained by reference to an appropriate Land Information System.

2.23 DATA VERIFICATION

By whom and date.

2.24 POSITIONAL UNCERTAINTY

Positional Uncertainty is the uncertainty of the coordinates or height of a point, in metres, at the 95% confidence level, with respect to the defined reference frame. (See Part A Section 4 for more information).

2.25 LOCAL UNCERTAINTY

Local Uncertainty is the average measure, in metres at the 95% confidence level, of the relative uncertainty of the coordinates of a point(s), with respect to adjacent points in the defined frame. This quantity supersedes ORDER, but ORDER may continue to be used until LOCAL UNCERTAINTY is fully implemented. (See Part A Section 4 for more information).

3. VERTICAL CONTROL DATA ELEMENTS

3.1 TYPE OF MARK

A full description of the mark is required including, where appropriate, material used, method of installation, presence of cover and marker post. Details about the stability of the mark should be included (e.g. black soil).

3.2 LOCATION

A sketch and/or written description providing sufficient information to locate the mark in the field and on index plans.

Recognising that a station may be situated in an environmentally, historically or politically sensitive area, the record should highlight:

- whether or not the station is in a sensitive area;
- the nature of that sensitivity;

- a reference to the responsible authority.

3.3 DATE OF HEIGHTING

All heighting techniques resulting in classifications of Class D or higher, may be included in the vertical data base. The date of completion of the heighting that has assigned the AHD value to the mark should be given.

3.4 NAME AND/OR NUMBER

This is designed to provide a unique identifier for the station. A number of identifiers may be attributed to a station ranging from complete alpha, complete numeric, to a combination of alpha numeric. Usually no more than two names are attributed to a station.

3.5 DATUM

The vertical datum to which the height of the mark is referred is to be defined e.g. AHD71, local.

3.6 LEVEL SECTION NUMBER

Where appropriate the reference number assigned to the levelling route between junction points which contain the bench mark, e.g. LIN 3218-4053.

3.7 CLASS

Alpha characters assigned to the survey.

3.8 ORDER

Numeric or alphanumeric characters assigned to the survey. ORDER has been superseded by LOCAL UNCERTAINTY, but ORDER may continue to be used until LOCAL UNCERTAINTY is fully implemented.

3.9 HEIGHT REDUCTION DOCUMENT

Details are required to identify the heighting or levelling adjustment from which the height of the mark has been determined.

3.10 HEIGHT

The height of the mark on the defined vertical datum should be given in metres and decimals (up to three) thereof.

3.11 DATE OF CURRENCY

This date is intended to reflect how up to date the information is. The currency date will be modified on the basis of the latest action on the data, whether it be in the form of reoccupation, additional observations, more recent adjustments etc.

3.12 RESPONSIBLE AUTHORITY

Indicates the authority responsible for the information including the State or Territory in which it operates.

3.13 FIELD RECORDS

An appropriate reference to the field records should be made including the authority that holds the records.

3.14 ADDITIONAL INFORMATION

Brief description of hardware and techniques used, along with any other information that may be of assistance.

3.15 PHOTO IDENTIFICATION

All bench marks should, where practicable, be identified on aerial photographs and details entered on the bench mark sketch plans.

3.16 PLANIMETRIC COORDINATES

Geographical or grid coordinates, to the best available accuracy, should be recorded. These can be scaled from appropriate maps, or determined by field survey.

3.17 POSITIONAL UNCERTAINTY

Positional Uncertainty is the uncertainty of the coordinates or height of a point, in metres, at the 95% confidence level, with respect to the defined reference frame. (See Part A Section 4 for more information).

3.18 LOCAL UNCERTAINTY

Local Uncertainty is the average measure, in metres at the 95% confidence level, of the relative uncertainty of the coordinates of a point(s), with respect to adjacent points in the defined frame. This quantity supersedes ORDER, but ORDER may continue to be used until LOCAL UNCERTAINTY is fully implemented. (See Part A Section 4 for more information).

4. DATA ARCHIVING POLICY

It is the data custodian's responsibility to ensure the appropriate safekeeping of survey observation and computation data. The resources allocated to this archiving depend on the importance of the data and must be addressed on a case by case basis. The overriding principle is that the more important the data, the more care should be taken in preserving it. When making an assessment the following items should be considered:

- Is the data of public interest
- Is the data of historic interest (the data may become useful for an apparently unrelated project)
- Is the data useful for future projects
- How much would it cost to replace the data
- How accessible should the data be (e.g. should hardcopy be scanned)
- In what format should the data be stored to make it easily used in the future (e.g. RINEX format for GPS data)
- On what medium should the information be stored (will the medium still be useable in the future,

and will it require maintenance in the meantime)
- Should the data have redundant archiving
- Should there be off-site storage in case of a major catastrophe.

5. GPS DATA ELEMENTS RECOMMENDED FOR ARCHIVING

5.1 STATION NAME

Any unique identifier can be used, but it is common practice to also assign a unique 4-character identifier that is used along with the year and day of year, as part of the GPS data filename.

5.2 GPS OBSERVATION ELEMENTS

Ideally this would include all the mandatory and optional fields in the Receiver Independent EXchange (RINEX) file (ftp://igscb.jpl.nasa.gov/igscb/data/format/rinex210.txt). but at least the information below should be available:

- The station identifier
- The observing authority
- The antenna eccentricities (in all three dimensions) -measured to the Antenna Reference Point (ARP).
- The receiver type
- The antenna type
- The GPS observables collected
- The satellites observed
- The observing times (start & finish)

A reference to the field records (including the original recording medium and any of its backups) should be made. The organisation holding those records should be mentioned.

5.2.1 Additional Information

Any other information that may be of assistance in analysing the results should be recorded. For example, restricted horizon or the type of external frequency standard that may have been used.

5.3 GPS SOLUTION ELEMENTS

There are many GPS software packages available and all producing similar results, though there is considerable variety in the information produced. Whatever software is used, the file(s) containing the results should be maintained to record necessary information about the process. Ideally this would include all the elements specified in the Solution Independent EXchange (SINEX) format (ftp://igscb.jpl.nasa.gov/igscb/data/format/sinex.txt), but at least the information below should be available:

- Processing Authority
- Software (name & version)
- Date & time of processing
- Unique solution identifier
- Data window
- Sampling interval
- Observables used
- Orbits used
- Solution options
- Ambiguity solution
- Station position(s)
- Variance-Covariance matrix
- Any additional comments

ICSM MEMBER ORGANISATIONS

The latest contact information for ICSM members can be found on the ICSM web site.

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