Connecting a Cadastral Survey to PNG94 using GNSS

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This is an essential workshop for surveyors intending to use GNSS techniques to connect their cadastral surveys to PNG94, as is now required by PNG survey legislation.

The workshop will cover the following topics in detail:

1. Legal traceability requirements for GNSS in Cadastral surveys in PNG
2. Finding the nearest PNG94 Control
3. Accuracy and precision requirements, Positional & Local Uncertainties
4. Which GNSS equipment/technique to use
5. Network design and observational procedure
6. Loop closures, fault finding and adjustment
7. Computations on PNGMG94
8. Computing combined Grid and Sea-level Corrections
9. Converting PNGMG94 Grid distances to Ground Distances
10. Worked example of a PNG94 Survey

These notes describe practical steps for surveyors who intend to connect cadastral surveys to PNG94 using Global Navigation and Surveying Systems (GNSS) such as the US Global Positioning System (GPS).

1 Connecting a Cadastral survey to PNG94

1.1 PNG94 and PNGMG94

PNG94 (The Papua New Guinea Geodetic Datum 1994) is a geocentric geodetic datum, gazetted by the PNG Government in 1996 to be used as the basis for all new land surveys in PNG. PNG94 is defined by a network of 14 fiducial (absolute trust) geodetic stations in Papua New Guinea surveyed by GPS between 1993 and 1994. The PNG94 coordinates of the 14 stations are defined by their International Terrestrial Reference Frame 1992 (ITRF92) coordinates at epoch 1994.0 (1st January 1994) (included in Appendix 1). This is the same realisation as GDA94 in Australia. 1994.0 is called the reference epoch. The reference ellipsoid used by PNG94 is the Geodetic Reference System 1980 (GRS80) Ellipsoid which has the following defining parameters:

GRS80 semi-major axis (a) = 6378137 metres, Inverse flattening (1/f) = 298.257222101

GRS80 is also the reference ellipsoid used by ITRF and GDA94. The GRS80 ellipsoid is practical identical to the WGS84 ellipsoid (although datum coordinates will be different) being only 0.1 mm different at the poles.

PNG94 has benefited from extensive research into active plate tectonics in PNG and over 140 geodetic stations have been built or reused to monitor tectonic deformation since 1993. The precise ITRF coordinates and site velocities of these stations have been a useful by-product of the research, and these stations have been used to densify the datum (Figure 1 and Appendix 1). High resolution modelling of tectonic plate motion in PNG94 means that coordinates at epoch 1994.0 (1st January 1994) can now be computed, mitigating the adverse affects of unmodelled tectonic deformation in extensive geodetic surveys.

Grid projection coordinates in PNG94 are referred to as PNGMG94 (Papua New Guinea Map Grid 1994) coordinates. PNGMG94 is a UTM (Universal Transverse Mercator) projection. WGS84 and ITRF2005 UTM parameters can be used in GIS and GNSS post-processing software interchangeably with PNGMG94 (GRS80 ellipsoid, False Easting 500000, False Northing 10000000, Central scale factor 0.9996, 6° wide zones).

The PNGMG94/UTM Zones in PNG are:

Zone 54 South   (from the Indonesian Border to 144 deg East),
Zone 55 South   (from 144 to 150 deg East - which is most of Central PNG), and
Zone 56 South   (from 150 deg East to the Solomon Islands).

Where a survey straddles two adjacent zones, the zone to be adopted for the survey should be the zone where the majority of land lies, to avoid the need to do zone to zone transformations of Grid coordinates.

Spreadsheets to convert between PNG94 Ellipsoidal coordinates (latitude and longitude) to PNGMG94 UTM Coordinates and compute grid convergence and scale factors are described in Section 6.

After processing and adjustment of the GPS survey has been completed, the coordinates and grid distances need to be transformed to the local ground based (topocentric) Cadastral Plane system for use in Cadastral or Engineering surveys. This process is discussed in detail in section 4.

All cadastral surveys should be connected to the nearest PNG94 control station with validated coordinates (figure 1 below and Appendix 1). The station must be located on the same tectonic plate as the survey area, otherwise significant errors will be introduced into the survey if these stations are used. If there are no validated control points within 50 km of the survey area then precise point positioning (PPP) methods such as NRCan or AUSPOS should be used in conjunction with a tectonic site velocity model (section 3.61). Surveying across plate boundaries is not recommended unless the baseline changes that will have occurred between 1994 and the date of the survey are known.
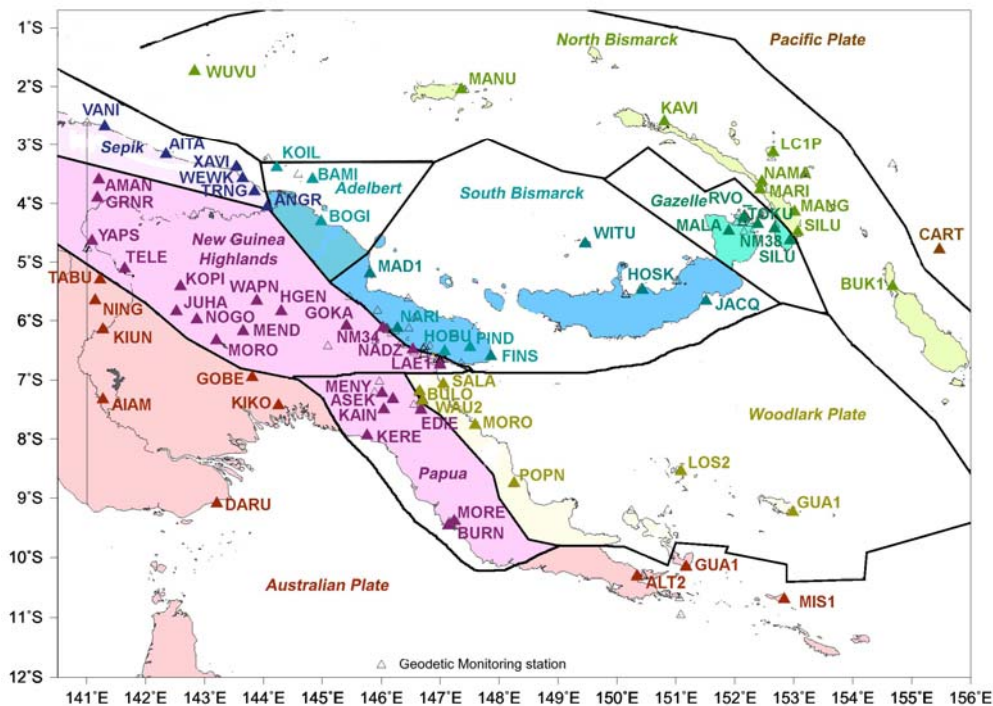

Figure 1. PNG94 primary geodetic control and plate zones

1.2   WGS84 and ITRF2005

WGS84 and the closely related ITRF are not legal datums for Cadastral Surveys in PNG, because there is no defined WGS84 datum point in the country and both are kinematic or dynamic datums (the coordinates change constantly with time). Therefore, WGS84 coordinates are not legally traceable to the PNG cadastral system. PNG94 is a static geodetic datum and PNG94 coordinates are frozen in time, whereas WGS84 and ITRF coordinates for survey control change by up to 10 cm per year due to the motion of tectonic plates in PNG. WGS84 and ITRF were in agreement with PNG94 only at the beginning of 1994. Since then, the

differences between PNG94 and WGS84 are now almost 1-2 metres or more (and increasing by up to 100 mm per year) with the difference and rate of change varying depending upon the location.

WGS84 and ITRF coordinates change constantly and the stations which define the WGS84 datum are very distant from PNG. Unless the connection to the external datum is done using precise orbits and the epoch of measurement is also included in the datum definition, it becomes impossible to relate different WGS84 surveys conducted at different times with any accuracy. For this reason WGS84 should only be used in PNG for navigation and 2-3 metre accuracy mapping surveys. Nevertheless, WGS84 ellipsoid parameters can be used in GNSS processing and RTK software as the dimensions are practically identical to the GRS80 ellipsoid used by PNG94 and the Grid projection parameters are the same.

1.3   What effect does tectonic movement within PNG have on PNG94?

Internal tectonic deformation within PNG is very rapid (up to 12 cm per year across some plate boundaries). (Figure 2). In addition, large earthquakes, landslips, soil creep and volcanic activity can cause displacements of up to several metres. Baselines between many of the 14 primary PNG94 stations have changed by up to 2 metres since 1994. This means that errors of the same magnitude are introduced into surveys if the tectonic movement is not modelled for surveys across plate boundaries.
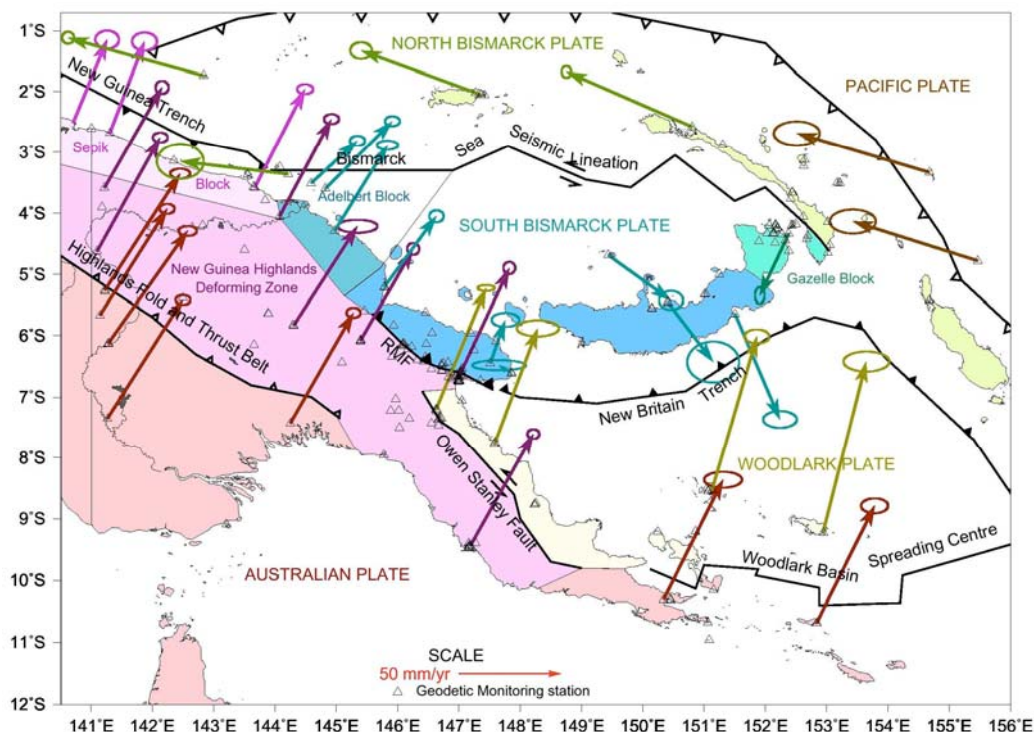

Figure 2. Annual tectonic motion of PNG94 stations within ITRF and WGS84

The guiding principle should be

"*The PNG94 coordinates of any point should describe where the point was in 1994, not where it is now*".

The reasoning behind this principle is that all surveys should be able to be connected together seamlessly using fixed ground monuments, within a local area, and this can only be achieved in a tectonically active environment by adopting a fixed reference epoch of a nearby control station in which to relate surveys to the parent datum (Figure 3).
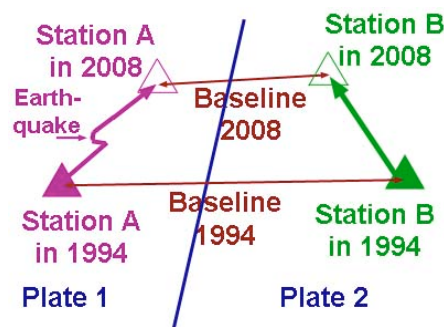

Figure 3. Effect of tectonic deformation on coordinates & baselines

1.4   Requirement to connect a Cadastral survey to PNG94

A modern Spatial Data Infrastructure (SDI) requires a seamless fit for different layers of spatial information for any given location or parcel of land. This situation can only be achieved by ensuring that the same geodetic datum is used by all the data used in the SDI, or at least the relationship between different (e.g. earlier) datums used in the SDI should be known by means of transformation parameters. For this reason, land administration authorities all over the world have been driving changes in survey legislation over the last 30 years to ensure that all cadastral surveys are connected to a geodetic datum with an acceptable level of Positional and Local Uncertainty (PU and LU). If the coordinates of each corner of a land boundary are survey accurate within a GIS, then other spatially dependant information such as survey plans, imagery (e.g. Google Earth), utilities, topography, roads can be related to these boundaries within the GIS. Previously, many survey plans were floating in space and it was difficult to integrate a survey plan into a GIS with any degree of accuracy with misfits of up to tens of metres and datum swings of several degrees. The PNG *Survey Act* and PNG *Survey Coordination Act* now require that Cadastral Surveys are connected and oriented to PNG94, to support a functioning SDI in PNG.

The PNG National Mapping Bureau has been progressively surveying cadastral PSMs around the country to enable older survey plans to be coordinated, so that PNG94 coordinates of land boundaries can be built up into a Digital Cadastral Database (DCDB), which underpins any GIS.

1.5   Benefits of using GNSS for Cadastral surveys in PNG

Traditionally, surveys have been undertaken using conventional terrestrial methods (i.e. using a theodolite or total station to measure angles; a steel band or EDM to measure distances). Line of sight was required between any two measured points in order to complete a survey. GNSS methods such as GPS have overcome many of the limitations that have restricted the use of conventional surveying methods. No line of sight is required to compute a baseline between two GPS stations, and baselines can be measured with centimetre precision sometimes over hundreds of kilometres. A GNSS/GPS receiver can be considered to be a long range total station / prism combination. The rugged terrain and remoteness of many locations in PNG makes GNSS the surveying method of choice in most situations.

There are still limitations with using GNSS however. First, GNSS derived baselines and positions vary in quality considerably from millimetres to several metres. To obtain good quality GNSS observations, at least several GNSS satellites should be visible. This means that any GNSS antenna has to have a clear view of the sky (at least 15° above the horizon), usually requiring clearing of vegetation and trees from nearby, and avoiding nearby buildings or overhanging objects. Some skill is also required to process GNSS baselines and assess their accuracy. Just as a survey traverse has to close using terrestrial methods, so too do loops of interconnected GNSS baselines. Old surveying principles still apply, even using 21st Century technology.

1.6   Positional Uncertainty (PU) and Local Uncertainty (LU)

Two new quality indicators have been devised to be used with GNSS surveys and GIS, Positional Uncertainty (PU) and Local Uncertainty (LU). The PU gives an estimate of how close the coordinates of a position are with respect to the Geodetic Datum (similar to the old "Order" classification). The LU gives an estimate of how close the coordinates of a position are to neighbouring points and local control (similar to the old "Class" classification). A station with a high PU (low accuracy) can be used for precision local surveys that will have low LUs (high relative precision), and conversely, tape and compass measurements (which result in high LUs (low precision)) can be used from a station of low PU (high accuracy). Spatial data and survey coordinates should have both PU and LU tabulated in all three dimensions.

Most coordinates on older survey plans are likely to have low LU, but high PU, because of the very high cost and difficulty of connecting a survey accurately to the geodetic datum before the GNSS/GPS era. To summarise, the PU is a measure of how close a survey plan is aligned with the Geodetic datum, and LU is a measure of how accurate internal plan measurements and boundary dimensions are.

1.7   What are the accuracy requirements for connecting to PNG94?

The *PNG Survey Directions, 1990* have been the regulatory document regarding the accuracy of land surveys in PNG. They were written before GNSS methods were widely used in cadastral surveys and don't define how closely a survey should be connected to PNG94 by providing measures of LU and PU. Table 1 outlines suggested uncertainties for Cadastral surveys in PNG. These have yet to be formalised.

Classification	Application	Suggested Positional Uncertainty (PU)	Suggested Local Uncertainty (LU)
Urban Class 1	Urban, residential, commercial	100 mm	30 mm
Rural Class 1	Land used for resource extraction, utilities, pipelines	300 mm	100 mm
Rural Class 2 (A)	smaller settlement blocks	1 m	300 mm
Rural Class 2 (B)	larger settlement blocks	2 m	500 mm
Rural Class 3	Customary Registration of rural land for individuals or families	10 m	3 m
Rural Class 4	Other Customary Land surveys	30 m	10 m

Table 1 PNG Cadastral surveying accuracy requirements (derived from *PNG Survey Directions, 1990*)

1.8   Legal Traceability (LT) for Cadastral Surveys

Traditionally, cadastral boundaries in the Torrens system have been defined by bearings and distances and related to a local datum line (defined by local survey control and reference marks). Bearings have either been arbitrary (usually based on local magnetic North), True (from astronomical observations), or Grid (e.g. from geodetic control).

The exact orientation of a parcel of land does not affect the dimensions or area of the parcel, and consequently there has been much variation in the orientation of different survey plans. Distances, however impact directly on the dimensions, utility and value of the land. Consequently, there have always been strict legal requirements for traceability of distances to international and national measurement standards in order to legally define the dimensions of a land parcel. For this reason, excess has been built into earlier surveys to provide protection to title holders against the risk of shortage. Unlike steel bands and tapes, EDM and GNSS methods do not physically measure a distance and so strict calibration requirements and measurement procedures have been put in place to ensure that distances measured using these methods are correct for the precision required for the survey. Another important consideration for GNSS is that baselines are usually reduced to ellipsoidal or projected Grid coordinates during data processing. GNSS Distances between two points are usually not the horizontal ground distances that are required for a Cadastral survey and a scale factor needs to be applied to convert these Grid or ellipsoidal distances to local Ground level.

Many land administrations are moving towards legal acceptance of geodetic coordinates to define land boundaries, especially as result of CORS networks being established. In PNG, this situation has been in place for some time for Petroleum and Mining Leases (which are legally defined by geographical ellipsoidal coordinates related to a datum e.g. AGD66 - Bevan Rapids). This has led to the concept of a legal coordinate datum. Procedures and checks have yet to be formally documented in amendments to the PNG Survey Directions specifying how a GNSS survey can legally define certifiable coordinates for a land parcel with respect to a datum. These workshop notes go some way to addressing this shortcoming.

The take home principle is that any GNSS cadastral survey in PNG should be connected to at least 1 PNG94 geodetic control point with a Positional Uncertainty (PU) lower (better than) than the value setout in Table 1. The relationship between adjoining corners of any land survey should also be surveyed with a precision (Local Uncertainty (LU)) better than that shown in Table 1. All coordinates and dimensions used for GNSS cadastral surveys should be derived from a closed loop of baselines, or by independent measurement (double check radiation on a different day, or from radiation from a different station).

2   GNSS equipment and procedures needed to attain specified PU and LU for Cadastral surveys

GNSS methods can deliver precision from millimetres to tens of metres, so some care needs to be taken over what sort of GNSS equipment and technique is used to undertake a cadastral survey. The minimum standard equipment and observing techniques are shown in Table 3.

The main GNSS methods are summarised as follows:

1. Post-processed Static GNSS observations
2. Single-Base Real-Time Kinematic (RTK) observations
3. Networked RTK observations
4. Differential GNSS/GPS (DGPS)
5. Precise Point Positioning (PPP)
6. Point Positioning

2.1 Post-processed Static GNSS observations

This is the most rigorous and preferred method for a GNSS survey where precision is required (e.g. for Urban surveys). A baseline can be estimated with millimetre to centimetre precision by differencing of carrier-phase observations made at both ends of the baseline with two geodetic grade GNSS receivers. Conceptually, the base station receiver/antenna acts as a total station, and the rover receiver/antenna acts as a prism. Provided that sufficient carrier-phase observations are made, then the baseline can be fixed (measured accurately with statistical significance). If there are insufficient observations, or the quality of the observations are poor (e.g. the antenna is next to a tree), then the baseline might not be fixed (e.g. a float solution, which is statistically uncertain), so a great deal of care needs to be taken.

In most instances at least two receivers are needed for a Static survey. If a Continuously Operating Reference Station (CORS) is within range of the survey area, e.g. NMB's MORE base, RVO's RVO_ base or Unitech's, LAE1 base, then a survey can be undertaken using a single receiver, provided that the base station is logging data and that the data are available for processing.

Single frequency (L1 only) geodetic grade receivers can usually solve mm/cm precision baselines up to 10 km. Longer baselines in PNG usually require a more expensive dual-frequency (L1/L2) frequency receiver to measure precision baselines up to 50 km using a broadcast ephemeris (satellite orbit information broadcast by the GNSS system) because single frequency differencing cannot model ionospheric delay. Beyond 10 km single-frequency receiver baseline precision starts to degrade rapidly. Surveyors working in rural areas should use dual-frequency receivers to achieve the required precision.

A Static baseline is measured by post-processing GNSS data collected by receivers at either end of the baseline (the receivers have to be observing at the same time). The length of time required for observing depends upon the length of the baseline, the satellite geometry and the quality of observing conditions (Table 2). If observing conditions are poor at any station, then the times recommended in Table 2 should be doubled or even tripled. Bad conditions include: nearby trees (especially pine and coconut), high grass, buildings, towers and periods of poor satellite availability or geometry (DOP) (use GNSS planning or prediction software to identify when these periods are). In addition, if there is more than 400 metres elevation difference or if humidity levels vary significantly between the base and reference station, then the times should be doubled as well in order to mitigate tropospheric modelling errors. In ideal conditions, single-frequency receivers can measure fixed baselines up to 20 km, but 10 km is the maximum recommended range in practice.

The cost of accessing a station location in PNG can be very considerable, for example if helicopters or boats have to be chartered to gain access, so it is strongly advisable to observe any station for as long as practically possible to increase the probability of a baseline fix, if there are any adverse observing conditions.

Baseline length	dual - frequency (minutes)	single - frequency (minutes)
0-5 km	15	30
5-10 km	20	40
10-20 km	30	60+ 50% chance
20-30 km	40	unlikely
30-40 km	50	
40-50 km	60	
> 50 km*	300	

Table 2. Occupation times for different baseline lengths (good observing conditions)
Times should be doubled or tripled if any station has adverse observing conditions.
* over 50 km requires PPP such as AUSPOS or NRCan or precise orbit.
If using AUSPOS start obs after 10:00 PNG Time if possible.

Loop closures and baseline redundancy

A baseline is essentially just like a traverse line or radiation measured by a total station. A GNSS baseline traverse or network should consist of closed loops of baselines and the closure (Loop closure) should be within the standards required for the survey. Likewise, important GNSS baseline radiations should be checked by repeat measurements on a different day, or by a check radiation from a different station (and the coordinates should agree within acceptable tolerances).

If a single receiver is being used with a CORS base station, then it is vital that any stations are surveyed twice (at least six hours apart, but preferably the next day at a different time of the day, so that a different configuration of satellites is observed) to ensure that an independent check is made.

Both single and dual frequency receivers can be used on the same survey. For example, the dual-frequency receiver can be used to measure the baseline from a PNG94 station or CORS station some distance away, to provide a local base station. The local survey can then use single frequency receivers provided the baselines are less than 10 km.

2.2 Real-Time Kinematic (RTK) surveys

RTK surveys provide real-time coordinates to surveyors in the field using GNSS receivers equipped with radios or mobile phones. The GNSS RTK base station sends instantaneous corrections to the rover receiver using a radio or mobile phone link. RTK is usually only accurate in open areas with good sky visibility and for this reason generally has limited application in PNG. Intervening topography also disrupts any correction signals, further hampering its application in typical PNG settings.

If RTK is being used for Cadastral surveys it is essential that:

1. Two or more nearby PNG94 control points are measured as a check (and that the coordinates agree) before any cadastral survey is commenced.

2. All recorded positions have **RTK-fixed** positions. Float solutions can be very unreliable.

3. All points are surveyed twice (on different days, or from different base stations) to ensure coordinate repeatability, and measured three or more times if necessary until consistent coordinates are given.

For example, if the initial survey was conducted on Monday afternoon, then the repeat (check) survey should be conducted on Tuesday morning, to ensure that a different selection of satellites are used.

The independent check is very important with RTK, as there is the possibility (~1:200) that even Fixed solutions are inaccurate, especially if observing conditions are poor. At the start of any RTK survey, it is essential to take the first measurement at another control station (2 if possible), in order to verify that the

datum, projection system, geoid model and antenna measurements have been setup correctly into the system. The difference between the RTK solution and the control should be less than 15 mm. If possible, other existing control stations should be included in any RTK survey wherever possible, to verify the performance of the system.

Both single and dual frequency GNSS receivers equipped with special radios can be used for RTK. The typical RTK ranges are 5 km for single frequency (L1 only) and 10-20 km for dual-frequency (L1/L2), provided that correction signals can be transmitted between the base and rover. More powerful radios and repeater stations can increase the range of the system.

A recent development is GNSS is Networked RTK (NRTK) where a network of base stations compute a combined network correction, broadcast by mobile internet to GNSS receivers. PNG presently has no NTRK service in operation, however the principles of independent checking will still apply whenever NRTK is implemented.

2.3 Differential GNSS/GPS (DGPS) (e.g. OmniStar)

Many services provide differential corrections (by radio or communications satellite) to subscribed users with varying degrees of precision. A popular service used in PNG is OmniStar. A stand-alone GNSS receiver equipped with OmniStar can achieve reasonable accuracy for many surveys anywhere in PNG using this service. Recently, OmniStar have started operating a base station in Lae, and the quality of the service has been significantly improved for PNG users. It is very important to note that OmniStar delivers ITRF2005 coordinates (not PNG94). As stated previously, ITRF2005 coordinates are not acceptable for cadastral surveys in PNG because of the 1-2 metre difference in coordinates from PNG94. A correction has to be applied to any OmniStar coordinates in order to obtain PNG94.

The most practical way to do this is to use OmniStar to measure a PNG94 control station within 50 km of the survey area, provided that the survey area and station are on the same tectonic plate. If the station is more than 50 km from the survey area, or on a different plate then a site velocity calculator will have to be used instead (refer section 3.62) The difference between the OmniStar ITRF solution (two repeat observations at the commencement of the survey) and the PNG94 tabulated value is then applied to all of the other OmniStar measurements. The PNG94 station is measured again at the end of the survey to "close" the observations. Ensure that the correct UTM Zone and Hemisphere are configured for the area.

The correction applied is as follows:

Easting correction = PNGMG94 Easting of control station - ITRF2005 UTM Easting from OmniStar
Northing correction = PNGMG94 Northing of control station - ITRF2005 UTM Northing from OmniStar

so that for each subsequent OmniStar measurement:

PNGMG94 Easting = ITRF2005 UTM Easting from OmniStar + Easting correction
PNGMG94 Northing = ITRF2005 UTM Northing from OmniStar + Northing correction

The correction will change every six months due to tectonic motion of the site and update of the OmniStar reference network coordinates.

Three main OmniStar service levels are available:
>	**OmniStar-HP** service delivers 100 mm precision,
>	**OmniStar-XP** service delivers 300 mm precision, and
>	**OmniStar-VBS** delivers 1 m precision.

With these levels of precision, OmniStar HP can be used for Rural Class 2A and above, OmniStar XP for Rural Class 2B and above, and Omnistar VBS for Rural Class 3 or 4 surveys. Again, it should be stressed that the

OmniStar has to be calibrated against a known PNG94 control station in order for the coordinates to be used for a PNG94 survey.

In order for OmniStar observations to be acceptable for PNG94 Cadastral surveys (Rural Class 2, 3 and 4), the following checks and corrections need to be done:

1. Two measurements need to be made on the nearest PNG94 control station at least six hours apart to determine the correction to be applied to other OmniStar coordinates for the survey (correction is PNG94 - OmniStar).

2. Each surveyed point must have a displayed sigma of $1/3^{rd}$ the tolerance for the survey (e.g. 10 cm on OmniStar display = 30 cm tolerance).

3. Each point must be surveyed twice not less than 6 hours apart (ideally on different days) and the coordinates agree within tolerances. (If not then the point will need surveyed more often until there is sufficient confidence in the solution)

2.4 Precise point positioning (PPP)

For some time, free online post-processing services such as AUSPOS and NRCan have been in operation. These services compute ITRF2005 coordinates for any given dual-frequency GPS observation data of 1 hour or more in RINEX format. The longer the observation time and better the observing conditions, the more precise the solution is. A 24hr observation period between 10:00 PNG Time (0:00 UT) and 10:00 the next day, usually provides acceptable results. The recommended minimum observation time is 6 hours observation (to get 2-3 cm precision). Observing times less than 5 hours are not recommended unless 10-20 cm accuracy is acceptable.

Unfortunately, these services do not transform the ITRF2005 coordinates to a local datum outside Australia and North America. If these services are used, then some care needs to be taken when converting the coordinates to PNG94. Because of the rapid tectonic deformation in PNG, there are no simple transformation parameters that can be used to do the conversion.

There are two transformation options available:

1. Do observations on a nearby PNG94 control point and apply the difference in coordinates (PNG94 - ITRF2005) to other stations that have used a PPP service within a 50 km radius (the same procedure as with OmniStar) (refer section 3.62).

2. Use a Plate motion calculator, to compute the ITRF2005 tectonic site velocity coordinates and convert the ITRF2005 coordinates to epoch 1994 (Section 3.62).

2.5 Point positioning

Point positioning is the technique used by most standalone GNSS receivers (e.g. handheld, or vehicle based). The accuracy varies from a few metres to tens of metres depending upon observing conditions and satellite geometry. While point positioning is useful for navigation, finding old survey control and low accuracy surveys, it is not acceptable under any circumstances for Cadastral surveys. The reason for this is simple.

1. There is no quality assurance for the coordinates displayed
   (i.e. the displayed coordinates could be significantly in error in 2D mode for example)

2. The positions are not traceable to a nearby geodetic datum monument.

Classification	PU	LU	Distance from nearest PNG94 control	Minimum Equipment	Minimum Technique
Urban Class 1	100 mm	30 mm	< 10 km	Geodetic Single Frequency GNSS	Closed baseline loop from PNG94 control or repeat RTK
Urban Class 1	100 mm	30 mm	> 10 km	Geodetic Dual-Frequency GNSS	Closed baseline loop from PNG94 control or repeat RTK
Rural Class 1	300 mm	100 mm	< 10 km	Geodetic Single Frequency GNSS	Closed baseline loop from PNG94 control or repeat RTK
Rural Class 1	300 mm	100 mm	> 10 km	Geodetic Dual-Frequency GNSS	Closed baseline loop from PNG94 control or repeat RTK
Rural Class 2 (A)	1 m	300 mm	< 10 km	DGPS enabled GNSS	Double-check radiation from PNG94 control
Rural Class 2 (A)	1 m	300 mm	> 10 km	DGPS enabled GNSS	Double-check radiation from PNG94 control
Rural Class 2 (B)	2 m	500 mm	< 10 km	DGPS enabled GNSS, OmniStar-HP	Comparison with nearby PNG94 control
Rural Class 2 (B)	2 m	500 mm	> 10 km	DGPS enabled GNSS, OmniStar-HP	Comparison with nearby PNG94 control
Rural Class 3	10 m	3 m	< 10 km	DGPS enabled GNSS, OmniStar-VBS	Comparison with nearby PNG94 control
Rural Class 3	10 m	3 m	> 10 km	DGPS enabled GNSS, OmniStar-VBS	Comparison with nearby PNG94 control
Rural Class 4	30 m	10 m	< 10 km	DGPS enabled GNSS, OmniStar-VBS	Comparison with nearby PNG94 control
Rural Class 4	30 m	10 m	> 10 km	1 DGPS enabled GNSS, OmniStar-VBS	Comparison with nearby PNG94 control

Table 3. Minimum standard GNSS equipment and technique for Cadastral survey

3 Static GNSS methods for cadastral surveying in PNG94

3.1 GPS Equipment required

If there is no validated PNG94 control within 10 km of the project area, then dual-frequency GPS receivers will be required. If the validated control is less than 10 km away, then single-frequency receivers can be used.

3.2 Identify and establish Cadastral Control for GNSS observations

Identify any existing reference marks or PSMs that are connected to earlier surveys in the area. If possible, these marks should be cleared of vegetation so that quality GNSS observations can be made. If there are no existing marks, or the marks aren't suitable for GNSS observations, then new stations (e.g. PSMs) will need to be constructed in open spaces. Good locations are nearby airstrips, helipads, playing fields, or open areas in government or company compounds. There should be at least three intervisible stations on the survey that are sufficiently far apart to be used as backsights for conventional surveys. Initially, a high accuracy datum station should be established (or adopted) reasonably central to the survey area, usually a PSM (plaque or star-picket set in concrete). GNSS stations should also be established within line of sight of older reference marks and PSMs that are in locations where GNSS observations aren't suitable, so that earlier surveys can be connected to the GNSS survey.

The basic idea, is that GNSS is used to establish control (and PNG94 azimuth) to be used as basis for conventional surveys using a total station or theodolite. In most surveys in PNG, it is usually impractical and unwise to attempt an entire survey using GNSS. A good strategy is to use GNSS to survey in a series of new (or existing) stations in open areas with good satellite visibility first. The stations don't have to intervisible, however it is a good idea to have some pairs of stations (or three) far apart that can be used as instrument and backsight stations for use as an azimuth datum for total station surveys.

3.3 Network and Baseline design

Using three receivers is recommended wherever possible. The advantages of using three receivers are:

1. Loop closures and checks are possible without re-occupation of marks
2. Logistically efficient and faster as it is usually not necessary to re-occupy the station if the correct procedures and occupation times have been used.

If only two receivers are available, surveyed points will need to be occupied twice (for radiations) for quality assurance. This can significantly increase travel times and logistical costs for the client.

3.4 Observation planning

The first task is to bring PNG94 into the survey area. This is achieved by running a base station at the PNG94 station, and using the second receiver to initially survey in two or three stations (1 central + 1 at each end of the survey) in the survey area. If the nearest PNG94 control is more than 50 km away, or is inaccessible, then run one of the receivers at the new datum station and the second receiver at another new station in the project area and use PPP (e.g. AUSPOS or NRCan) to get coordinates. Wait two days, so that the Rapid Orbit Solution is obtained. The ITRF2005 coordinates will need to be converted to PNG94 (Refer to section 3.62).

If observing conditions are poor at any station, then the times recommended in Table 2 should be doubled or even tripled. In ideal conditions, with long observations, a pair of dual-frequency GNSS receivers can measure fixed baselines up to 50 km using the broadcast ephemeris, but 30 km is the maximum recommended range in practice. It is better practice to use shorter lines rather than longer lines, as a more reliable fixed integer solution is obtained.

Good Observing conditions:

> At least 7 satellites in view (preferably 9 or more) above 10 degrees elevation
> PDOP less than 3.0
> No nearby trees (especially pine), high grass or buildings (above 10 degrees elevation)
> No hills above 10 degrees elevation
> Difference in elevation between base and rover less than 400 metres

Poor observing conditions

> Between 4 and 6 satellites in view above 10 degrees elevation
> PDOP greater than 3.0
> Nearby trees, high grass or buildings
> Nearby hills above 10 degrees elevation
> Difference in elevation between base and rover greater than 400 metres

During poor conditions, longer periods of observations are required, ideally when satellite availability is optimal. Re-occupation of the station may be required if a fixed solution is not obtained initially.

Bad observing conditions

> Less than 4 satellites in view
> Overhanging trees, building eaves, structures (e.g. drilling rigs)
> Only a small section of sky is visible from GPS antenna

It is often impossible to gain any useful GPS observations during bad observing conditions. This means that observations have to made for very long periods when satellite availability is optimum. It is a good idea to establish two eccentric stations that are intervisible nearby a station with bad GPS visibility, that have better observing conditions. Use conventional survey techniques to coordinate the covered station from the nearby stations with better observing conditions.

3.41 Surveys with three receivers

Two base station / one rover setup
Two receivers are used as base stations and the third receiver is used as a rover. This a good configuration for single parties surveying multiple stations that are not intervisible (e.g. survey control densification and local reference stations).

One base station / two rover setup
One receiver is used as a base station and the two other receivers are used as rovers. This is a good configuration for new stations that will be used for conventional surveys (where one station will be used as an instrument station and the other as a backsight or azimuth station). It is important that both rover receivers have common observing times to enable the baseline between them to be computed to survey accuracy. The pair of stations should be at least 100 m apart (and as long as practicable) to minimise errors in azimuth estimation.

3.42 Survey methods with two receivers
It is very important that redundancy (checking) is done when two receivers are used as logistical constraints often prevent important checks from being done.

Radial survey from single base station
One receiver is used as a base station and the other receiver is used as a rover. The rover measures all stations radially (like total station radiations). Once all radiations are measured, the survey should be repeated using a different base station on a different day if possible (e.g. one surveyed in the first run of radiations). Baselines should be measured directly where any pair of stations may be used as total station / backsight pairs.

Leapfrog survey (a bit like a link traverse)
This is recommended for long alignment style surveys (e.g. pipelines, powerlines, roads). One receiver is started at the first station at the start of the alignment and the other is setup at the next station along the alignment. Once enough data is measured at both stations to fix the baseline, the receiver at the start is shutdown and moved to the third station along. The receiver at the second station can be kept running. Once enough data is gathered at stations 2 and 3, the receiver at 2 is shutdown and moved to station 4 and so on. This is similar in concept to a total station traverse.

3.43   Surveys with single receivers
It is possible to do a survey with a single receiver using a PPP service such as AUSPOS, and then converting the ITRF2000 or ITRF2005 coordinates to PNG94 using the tectonic site velocity (section 3.62). Several hours of observations are required to get a solution accurate to a few cm. A full day 18-24 hrs observations usually attains centimetre accuracy or better.

3.5   Observation procedure
Setup the base receiver (the receiver with the most memory and battery capacity) on the new datum station. Setup the roving receiver at the first of the nearest existing PNG94 control stations and start the receiver. Gather sufficient observations (see table 2 above) to obtain a fixed baseline solution, shut down the receiver and move to the second nearby control station. Repeat the process. Measuring two baselines from existing control to the new station provides a good degree of quality assurance.

3.51   Antenna setup procedure
1.   Cut down or trim any nearby branches, shrubs or high grass. Nearby trees and
      buildings adversely affect GNSS measurements
2.   Check the centering of the tribrach over the mark with a plumb-bob, or if a pole is
      being used, check that the spot-bubble is calibrated.
3.   Carefully measure the antenna height (3 times around the rim of the antenna) and
      make a detailed note of where the height has been measured to
      (take a photo if possible)
4.   Start the receiver and make a note of the start time in the log.
5.   Take a photo of the mark being observed (if possible)

3.52   GNSS log sheet (Figure 4) or field notes must contain the following info:
Station number, Date and start time of observations (PNG time) & end time, Antenna height, Sketch of point on antenna where height has been measured to, Antenna type and serial number, Note any abnormal situations (e.g. bad observing conditions, unstable mark etc..).

GPS Occupation Log		Antenna sketch
Site ID or filename		show point where measurement
Station Name		to antenna is taken
Antenna type		
Antenna serial number		
Height measurement (start)		
Height measurement (end)		
Height to Phase Centre		
Date start		
Time start (PNG Time)		**Approximate position**
Time end (PNG Time)		Latitude
Date end		Longitude

Figure 4. Example of minimal GPS observation log

3.53   Antenna height measurement

GNSS measurements are gathered at the phase centre within the antenna. This is the equivalent of the trunion axis of a theodolite, so the phase centre height has to be known in order compute relative heights using GPS. Different antennas may be used on the same job, so that antenna height measurements need to be converted to phase centre heights (true vertical) before the data are processed. Unless the base of the antenna (which is also usually the Antenna Reference Point or ARP) can be measured to directly (e.g. pillars, poles) then a slant height has to be measured to some point on the edge of the antenna.
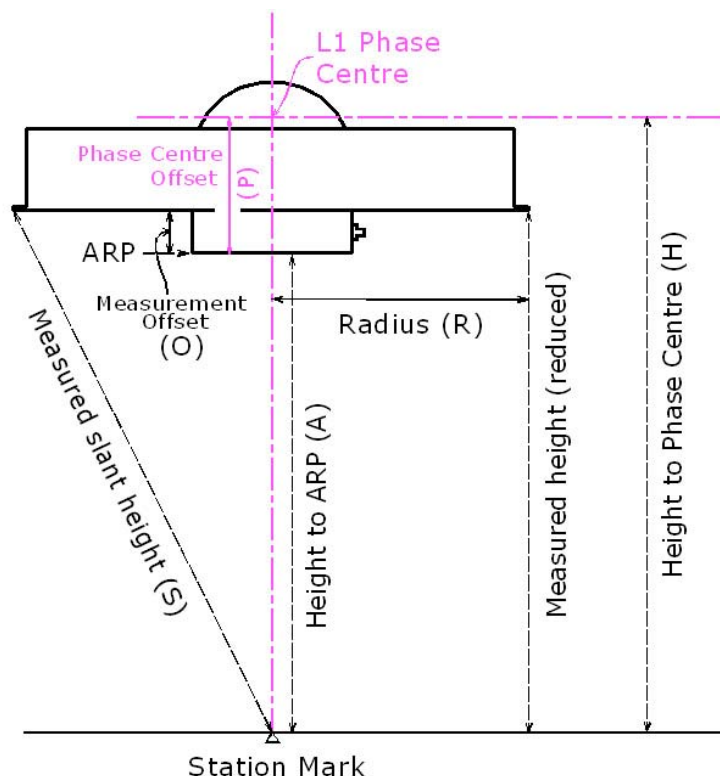

Figure 5. Antenna height measurements

Converting measured slant heights (S) to ARP heights (A)

$$A = \sqrt{S^2 - R^2} - O \quad \text{(Eq. 3.530)}$$

Converting slant heights (S) to Phase Centre heights (H)

$$H = \sqrt{S^2 - R^2} - O + P \quad \text{(Eq. 3.531)}$$

Where:

A is the height of the antenna reference point (ARP)

S is the slant distance from the ground mark to a point on the edge of the antenna

R is the radius of the antenna (radial distance from measured point to central axis of the antenna)

O is the vertical offset of the measured point from the base of the antenna (or ARP)

P is the phase centre offset (usually L1) or distance from ARP to the L1 Phase Centre

Formulae for commonly used antennas:

Ashtech Chokering (edge chokering) $H = \sqrt{S^2 - 0.19^2} - 0.035 + 0.11$

Ashtech Whopper (ASH700718a) (notch at top) $H = \sqrt{S^2 - 0.174^2} - 0.064 + 0.097$

Ashtech Geodetic (ASH700228D) (notch at top) $H = \sqrt{S^2 - 0.132^2} - 0.054 + 0.097$

Trimble 4000SSE w/ ground plane (notch at top) $H = \sqrt{S^2 - 0.233^2} - 0.059 + 0.063$

Sokkia GSR2700IS (outer lower rubber ring) $H = \sqrt{S^2 - 0.114^2} - 0.070 + 0.096$

Sokkia Radian IS (outer lower rubber ring) $H = \sqrt{S^2 - 0.114^2} - 0.124 + 0.140$

If the antenna is on a pole, the ARP height is usually the direct measurement from the base of the antenna to the end of the pole (A). To get the phase centre height, the L1 phase center offset (P) needs to be added on so: $H = A + P$ (Eq. 3.532)

3.6   Data Processing

The data from any receivers have to be downloaded. If different receiver models and manufacturers (i.e. different file formats) are used in the observing session then the raw data should be converted to Receiver Independent Exchange Format (RINEX). Any GPS data submitted to AUSPOS or NRCan must be in RINEX format.

Antenna slant heights need to be converted to Phase Centre heights (using equations in Section 3.53), or if the processing software has antenna models included, the heights only need to be converted to Antenna Reference Point heights. In practice, it is better to use Phase Centre Heights if the Phase Centre Offsets are known for the different antennas used.

If the base station coordinates are unknown, use a PPP post-processing package such as AUSPOS. If the base station coordinates are validated zero order PNG94 coordinates, then use proprietary baseline processing packages to compute PNG94 using the tabulated data.

3.61   Using AUSPOS or NRCan

RINEX data files can be directly uploaded to
Geoscience Australia's AUSPOS: ***http://www.ga.gov.au/bin/gps.pl***
or NRCan's service: ***http://www.geod.nrcan.gc.ca/online_data_e.php***

NRCan has a faster turnaround time than AUSPOS and also provides ITRF2005 coordinates in UTM. If possible, use both services to compare the results.

If the height to the Antenna Phase Centre is known, this should be entered in as the height with the "Default antenna" selected. Select the ITRF option for NRCan (the default is NAD-83 which cannot be used in PNG).

A minimum of one hour observations are required for an AUSPOS/NRCan solution, however in practice at least 6 hours (preferably as many hours as possible) of observations should be submitted to get a 2 cm accurate solution. Below three hours, the computed position can be uncertain, especially if observing conditions, DOP and number of satellites visible are poor.

It is best to wait 2-3 days after the date of observations to submit data to AUSPOS/NRCan to ensure that IGS data from nearby stations and IGS Rapid orbits are used in the processing. If data are submitted too soon, the Ultra-Rapid orbits are used and the solution might not be sufficiently accurate. The IGS Final orbit is available 2-3 weeks or so after the observations, and if possible the solution using this orbit should be used to achieve the highest accuracy.

Between 20 minutes and a few hours after submitting the data (depending upon the quantity of data submitted and the number of other users), a coordinate report is generated and submitted by email. The coordinates are provided in terms of both GDA94 and ITRF. At the end of the report an estimate of precision for the supplied coordinates is shown. If the precision is less than 15 mm, the solution is usually reliable, although the more realistic precision is typically between 2 and 3 times the indicated values. **The GDA94 solution must not be used in PNG** because most of PNG is not on the stable part of the Australian Plate.

3.62 Converting ITRF2005 coordinates to PNG94

ITRF coordinates (in Cartesian or Ellipsoidal format) need to be converted to PNG94 using a site velocity model. The correction should be in the order of 1-2 metres. A plate motion model for PNG is available from Paul Tregoning at the Australian National University http://rses.anu.edu.au/geodynamics/gps/png/vel.html
Richard Stanaway ***richard.stanaway@quickclose.com.au*** can also be contacted to obtain up-to-date site velocities in many parts of PNG. Refer to section 6 for spreadsheets to convert ITRF2005 coordinates to PNG94 with a known site velocity. The EGM96 model is used by AUSPOS to estimate MSL elevations for submitted data outside Australia. Remember that MSL (Height above geoid) height will need to be corrected to tie it in to an existing vertical datum if AUSPOS is being used on an existing survey.

Computing the epoch of measurement (Y_M)
The day of year (*doy*) of the survey is the number of days elapsed since the start of the year (e.g. 1st January is *doy* 1, and 31st December is usually *doy* 365 unless it is a leap year). The *doy* can be obtained from a Julian Day calendar, or more simply from the Rinex file submitted (e.g. RINEX file MORO0621.07o the day of year is 062 (number 5 to 7 in the filename))

The measurement epoch (Y_M) is the decimal of the year *= year + doy/365* for a standard year
e.g. 3rd March 2007 epoch = 2007 + 62/365 = 2007.170

PNGMG94 coordinates can be computed from AUSPOS using the following equations:

$$E_{PNGMG} = E_{UTM(ITRF)} + V_E(1994.0 - Y_M) \qquad \text{(Eq. 3.620)}$$

$$N_{PNGMG} = N_{UTM(ITRF)} + V_N(1994.0 - Y_M) \qquad \text{(Eq. 3.621)}$$

where,
E_{PNGMG} is the PNGMG Easting
N_{PNGMG} is the PNGMG Northing
$E_{UTM(ITRF)}$ is the ITRF UTM Easting derived from the AUSPOS solution
$N_{UTM(ITRF)}$ is the ITRF UTM Northing derived from the AUSPOS solution
V_E is the site velocity Easting component
V_N is the site velocity Northing component
Y_M is the epoch of measurement

Note: If the site has been displaced by earthquake or volcanic activity, then additional corrections are required to account for this.

The spreadsheet ***wgs84itrf_pngmg_convert.xls***
or http://www.quickclose.com.au/wgs84itrf_pngmg_convert.xls
can be used to convert WGS84 and ITRF2000/ITRF2005 UTM coordinates to PNGMG94 Grid coordinates with a known ITRF site velocity (differs significantly at each location beyond 10 km).

The ITRF site velocity in metres per year is entered into cells B4 and B5. The Year of the WGS84 or ITRF determination is entered into cell D4 and the Day of Year into cell D5. The dates are used to determine how much the site has moved in WGS84 /ITRF since the beginning of 1994. Copy row 10 down for as many conversions are required. WGS84/ITRF UTM coordinates in the local area are then copied into columns 1 and 2 and the equivalent PNGMG coordinates are computed and displayed in Columns 3 and 4.

3.63   Baseline post-processing
Most proprietary GPS baseline processing packages (e.g. GPPS, Trimble Geomatics Office, Sokkia Spectrum Survey etc..) initially compute baseline vectors between GPS antenna phase centres using a sequence of differenced code observations, triple differenced carrier phase and finally the double differenced carrier phase solution. The usual steps for processing a typical observing session in any baseline processing software package are:

Start a new project and setup the coordinate and height system
(It is recommended to use WGS84 UTM Southern Hemisphere processing parameters in PNG). Ensure that the UTM Grid zone matches the project area e.g.

Zone 54 South between the Indonesian border and 144° E
Zone 55 South between 144° E and 150° E
Zone 56 South between 150° E and 156° E

(For surveys which straddle two zones, the zone adopted is usually the one where most of the survey lies,
If a choice of geoid model is available, select EGM2008 or EGM96.

Load in the GPS data

(either raw data in a format compatible with the processing software, or RINEX). If the raw data format is not compatible with the processing software the data will be have to be converted to RINEX format either using software supplied with the receiver, or using the teqc program available from *http://facility.unavco.org/software/teqc/teqc.html* . Ensure that the loaded rover receiver data coincides in time (using table 2) with that of the base station. It is usually better to process baselines radially (in a single processing session) from a single base station, unless you have some confidence using the software's network adjustment utilities.

Fix the known station coordinates

Copy the coordinates from the master PNG94 control listing, or AUSPOS solution converted to PNG94 into the base station data entry in the processing software. In PNG it is better to enter the known ellipsoidal height for the reference station. If there is a space for orthometric heights (i.e., RL, MSL or height above geoid) leave this blank, or leave a "?" . If the known coordinate data is in ellipsoidal (latitude and longitude) format, use the the png94 calculator spreadsheet to convert the ellipsoidal coordinates to PNGMG Grid coordinates.

Preferably, copy and paste known PNGMG coordinate data from a spreadsheet or text file into the fields and overwrite the annoying ? mark. If you enter data manually, be very sure you enter the correct data and double check it.

If the fixed station has an accurate elipsoidal height, leave the elevation field (MSL) with a ?

Type in the ellipsoid height into the Height field.

If only an MSL elevation is known (i.e. no ellipsoid height), then type the MSL elevation into the elevation field and leave the height field with a ? in it.

Click on the down arrow next to the "?" box and Select "Control Quality" for the Easting/Northing and do the same for the height or elevation (which ever is known). Blue triangles appear to indicate that the coordinates are fixed. Close the properties box.

Run the baseline processing

Most packages generate a report of the baseline processing and reduced coordinates. Once the processing has finished, open these reports and review them.

3.64   Baseline processing assessment

If enough data has been gathered at each end of the baseline and there were no problems encountered in the baseline processing, the following conditions should be met for each baseline processed (exact description depends upon package used):

L1 fixed, narrow-lane fixed, L1/L2 fixed, ionospheric free fixed.

A fixed solution is usually reliable. Dual-frequency observations on short baselines are usually processed in single-frequency mode as higher precision is gained this way up to 5-8 km.   Be very wary of "float" solutions and especially differenced pseudo-range code solutions. Float solutions indicate that the baseline is most likely to be fixed, but there is a distinct possibility that it has been incorrectly fixed, in which case the baseline measurement could be in error. Code only solutions are usually only accurate to between 40 cm and a metre or so.

If a float solution is shown, then it is highly recommended to reobserve that baseline for a longer period during better satellite conditions (low DOP and more satellites) and to improve site observing conditions if possible. If the repeat measurement is also a float solution and is within 40 mm of the first measurement, it is usually safe to mean the two values.

If a code only solution is shown it is essential to reobserve the baseline for a much longer period and ensure that site and satellite conditions are better.

Things to look for with baseline statistics
RMS (root mean square) should be less than 10 mm for L1 fixed, 15 mm for ionospheric free fixed (L1/L2 fixed) up to 20 km, and 30 mm up to 50 km.
Reference Variance should be close to 1. If the variance is high (above 10) it indicates poor observing conditions
Ratio 1:n - The higher the ratio the more reliable is the fixed solution. In PNG if the ratio is lower than 3 it is worth reobserving the baseline, even if the report shows "fixed". The ratio is the difference between different double-differenced carrier phase solutions in the session. If the ratio is low, the chance of an incorrect fix is high.

If the baseline processing report is favourable and all baselines are fixed with RMS < 30mm, Ratio > 10 and Variance < 10) then the next step is to independently check the accuracy of the solution. This is usually done by remeasuring baselines to each new station from a different base station. The baseline processing should be repeated and the coordinates compared with the initial solution. This can be done by combining the baseline solutions in a network adjustment, or by comparing/meaning the different solutions in a spreadsheet.

If large differences (greater than the tolerance of the survey) are noted between two fixed solutions, checking of the loop closures in order to isolate any problem baselines is vital and make repeat or new measurements if necessary.

3.65   Check Loop Closures
Loop closures should be checked before any results are used. A small loop misclosure usually verifies that baselines in the loop have been fixed correctly. A large loop misclosure indicates that one or more of the baselines has been fixed incorrectly (or not fixed at all). If the loop closure is outside tolerances, then one or more baselines will need to measured again.

3.66   Adjust the survey
Once loop closures have been checked, then use the post-processing software to adjust the network. The Reference Variance Factor/Ratio should be close to 1. If the reference variance is high, then this indicates that one or more poor baselines have been overweighted (i.e. the baseline processing has overestimated the precision). These baselines should be weighted down and the adjustment repeated until the ratio is close to 1. If the ratio is below 1, then the baseline precisions have been underestimated. This isn't common with the GNSS processing though.

3.67   View report of adjusted coordinates
The adjusted survey coordinates should be viewed and the estimated errors compared with the PU and LU requirement for the survey. These coordinates can be copied and pasted into a text file or report.

3.68   Checking GNSS baseline radiations
If a baseline isn't part of a loop, then two baselines should be measured to the point and the coordinates compared and averaged. The difference in the coordinates should be less than the precision required for the survey.

3.7   Obtaining MSL values
While Reduced Levels (RLs) are not generally required for Cadastral purposes, they are required to compute height corrections to reduce GNSS distances to ground distances and for later engineering work.

Once the baseline processing has been finalised, the next step is to estimate the MSL elevations of the stations. If a geoid model (such as one derived from EGM96) is built into the GPS processing software (it is in AUSPOS), then the coordinate report will show both ellipsoidal heights and height above geoid using the inbuilt model.

EGM96 derived MSL can differ from true MSL by up to a metre in PNG, however for inland surveys that do not have accurate existing levels, using EGM96 is usually an acceptable strategy. In coastal areas, any PSMs

at tide gauges or other 1st to 3rd order MSL stations should be observed by GPS in order to estimate any offsets between EGM96 and local height systems and this constant difference should be applied to all other EGM96 derived heights in the survey.

For example, AUSPOS is used to obtain coordinates and EGM96 elevations of a PSM next to a tide gauge in Lae. The MSL elevation of the PSM is likely to be 1st order due to its proximity to the tide gauge. The EGM96 derived elevation of the PSM is 2.15 metres, however the MSL RL has been determined as 1.86 metres.) 0.29 metres needs to be subtracted from EGM96 derived elevations in the vicinity of the tide gauge to relate EGM96 derived heights to local sea level.

If an earlier height datum has been established in the project and is still used for height determinations, then the EGM96 derived height for the existing height datum station should be compared with the tabulated value in order to estimate the local offset to be applied to EGM96.

If the GPS processing software does not include a geoid model, then the separation between the ellipsoid and geoid (N value) needs to be obtained using the internet or other software packages. EGM96 N values can be obtained by entering site positions (latitude and longitude) into:

http://earth-info.nga.mil/GandG/wgs84/gravitymod/egm96/intpt.html

Another earlier software package, the PNG geoid program developed Professor Bill Kearsley of UNSW can extract N values in PNG. This program is available from the National Mapping Bureau and the Department of Surveying and Land Studies at UniTech, and requires a Windows 98 or earlier operating system to run. PNG Geoid values are of equivalent accuracy to EGM96 in the PNG Oilfields and the Markham Valley, but are less accurate (up to approx 2-3 metres) elsewhere in PNG because they are based upon an earlier global gravity model OSU91.

To convert ellipsoidal heights to MSL heights, use the following expression:

$$MSL_{EGM96} = h - N_{EGM96} \qquad \text{(Eq. 3.71)}$$

or,

$$MSL_{PNG} = h - N_{PNG} \qquad \text{(Eq. 3.72)}$$

where,

MSL_{EGM96} is the elevation above Mean Sea Level using the EGM96 geoid calculator
MSL_{PNG} is the elevation above Mean Sea Level using the PNG geoid calculator
h is the ellipsoid height and
N_{EGM96} is the N value (ellipsoid/geoid separation derived by EGM96)
N_{PNG} is the n value (ellipsoid/geoid separation derived by the PNG geoid program)

If GPS measurements are made at an existing height datum station then use the following expression:

$$MSL_{local} = h - N_{EGM96} + c \qquad \text{(Eq. 3.73)}$$

where,

MSL_{local} is the elevation above Mean Sea Level (in terms of existing local datum)
h is the ellipsoid height,
N_{EGM96} is the N value (ellipsoid geoid separation using EGM96), and
c is the local height datum correction
$(c = MSL_{localdatum} - MSL_{EGM96})$
where $MSL_{localdatum}$ is the fixed local height datum value

4   Establishing a Cadastral Plane datum

Ellipsoidal and Grid coordinates that are computed by GNSS are nearly always unsuitable for cadastral, feature surveys, engineering design, construction setout and as-built surveys. The reason for this is simple. Because of the necessary distortions involved projecting the earth's surface (which is spheroidal) onto a flat surface or cylinder (projection surface or plane), distances on the grid projection plane can be significantly different to the equivalent distances on the ground. The difference (scale factor) increases the larger the area of the earth's surface that is projected (Figure 6). Cadastral and engineering surveys require coordinates to be as distortion free as possible, so that ground distances, design and measured distances match those of the plan as close as possible to a scale factor of 1. For this reason, a projection system for these surveys should be centered on the survey area to ensure a scale of 1 can be used. Projection systems covering large areas such as UTM (including PNGMG94 and AMG66) have scale factors varying from 0.9996 at the central meridian to 1.0004 at the edge of the zone. A kilometre line on the ellipsoid surface (about 70 metres below sea level in PNG) would be 999.6 metres and 1000.4 metres long on the grid projection at these locations respectively. The higher one gets above the ellipsoid surface and sea-level, the smaller the scale factor gets. For example at an elevation of 3000 metres on the central meridian, the scale factor becomes as small as 0.999129, so that a 10 metre square building on the ground would be only 9.991 metres square in PNGMG94!

Many cadastral surveys in PNG have used UTM/PNGMG94 coordinates with a scale factor of 1. **This is a very bad practice**, as local Plane coordinates resemble Grid coordinates, even though they are not. Plane coordinates should be distinguishable from grid coordinates (for example by being a lot smaller in magnitude) to prevent confusion.

4.1 Computing the Combined Grid and Height scale factor.

Even if a local Plane Datum is used for a Cadastral survey, the Combined Grid and Height Scale Factor (or Combined Scale Factor) (k_p) will need to be determined and is usually required to be shown on any survey plan. Because of the high geoid-ellipsoid separation in PNG, The ellipsoid height should be used when computing the combined scale factor, as using an MSL elevation will introduce an error into the scale factor which can be significant over long distances. Refer to section 6 that describe spreadsheets for computing scale factors.

If the ellipsoid height is unknown for the survey area, then the N value (section above) can be added to an MSL elevation to obtain the ellipsoid height. Alternatively, 70 m can be added to any MSL value to get an approximate ellipsoid height (+/- 10 metres) anywhere in PNG, which reduces the error in estimating the scale factor if a geoid model isn't handy.
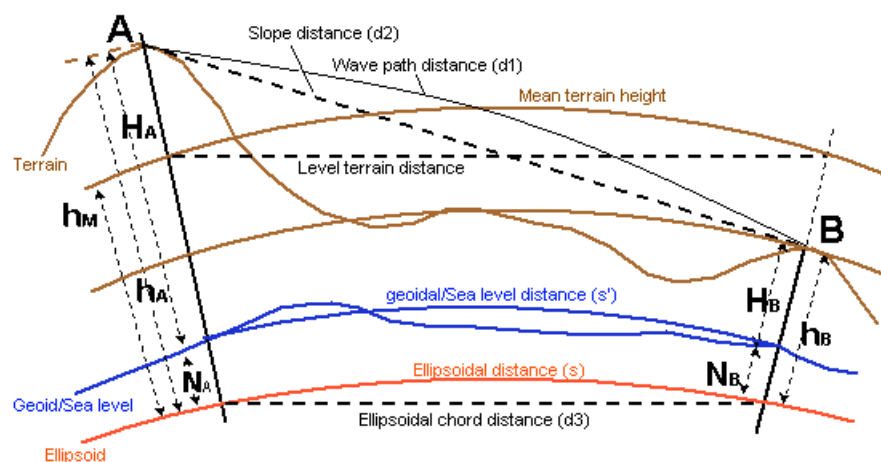

Figure 6. Difference between different distances

4.2   How to derive a Plane Grid System from PNG94

A local datum origin (can be a survey station, or a virtual point in space) should be chosen close to the centre of the project area (particularly in terms of Eastings). A height datum plane should be chosen that is close to the average elevation of the project area.

For example, a cadastral survey in the Highlands of PNG is bounded by E 520000 and E 540000, N 9060000 and N 9080000 and has a minimum elevation of 1600 metres and a maximum elevation of 2200 metres. The best fitting local Plane grid would have a central meridian of 530000 (mean of survey bounding eastings), a Northing origin of 9070000 and a height datum plane of 1900 metres (mean of survey area height). At this location, the scale factor would be 1 exactly. The further one gets away from this location (in terms of Eastings and elevation), the further the scale factor differs from 1.

In this example, the local coordinate origin would have PNGMG coordinates of
        E 530000 N 9070000 RL 1900
The local Plane coordinates of this location could be
        E 30000 N 70000 RL 1900
        (by subtracting 500000 from Eastings & 9000000 from Northings)
Bearings in Plane should be the same as PNGMG94 bearings, to make transformations easier.

Using a Plane coordinate system not only keeps scale factors close to 1, but the magnitude of the coordinates is easier to manage, with a smaller chance of typographical errors occurring (e.g. when entering coordinates into total stations etc.).

The combined PNGMG94 and height scale factor (k_p) at the datum origin would be 0.9993127 in the example above, so to convert between PNGMG94 coordinates and Plane coordinates the following expressions can be used:

$$E_{PLANE} = E0_{PLANE} + \frac{1}{k_p}(E_{PNGMG} - E0_{PNGMG}) \qquad \text{(Eq. 4.21)}$$

$$N_{PLANE} = N0_{PLANE} + \frac{1}{k_p}(N_{PNGMG} - N0_{PNGMG}) \qquad \text{(Eq. 4.22)}$$

$$E_{PNGMG} = E0_{PNGMG} + k_p(E_{PLANE} - E0_{PLANE}) \qquad \text{(Eq. 4.23)}$$

$$N_{PNGMG} = N0_{PNGMG} + k_p(N_{PLANE} - N0_{PLANE}) \qquad \text{(Eq. 4.24)}$$

where,

E_{PLANE} & N_{PLANE} are the local plane coordinates
E_{PNGMG} & N_{PNGMG} are the PNGMG coordinates to be converted
$E0_{PLANE}$ & $N0_{PLANE}$ are the Plane coordinates of the Plane datum origin
$E0_{PNGMG}$ & $N0_{PNGMG}$ are the PNGMG coordinates of the Plane datum Origin
k_p is the is the combined PNGMG Grid and Height scale factor at the Plane Origin

4.3   Converting a spreadsheet of PNGMG Coordinates to Local Plane

The spreadsheet ***pngmg_plane_convert.xls***
(http://www.quickclose.com.au/pngmg_plane_convert.xls)
can be used to convert PNGMG coordinates to a local plane system. Open the worksheet PNGMGtoPlane

1.   The PNGMG Easting and Northing coordinates of the Plane Grid origin are entered into the beige cells B4 and B5. The ellipsoid height if the origin is entered into B6. The combined grid and scale factor for the origin is computed and shown in cell F3.

2.   The chosen Plane coordinates of the same origin station are entered into cells D4 and D5.

3.   Row 10 is copied down for as many stations are required.

4.   PNGMG coordinates are copied (from GPS results) into Columns 1 and 2 and the equivalent Plane coordinates are computed and displayed in Columns 3 and 4.

To convert Plane Grids coordinates to PNGMG use the next worksheet (Plane to PNGMG)

4.4   How far can a Plane Grid be extended in order to satisfy cadastral and engineering tolerances?

Assuming that a maximum error of 20 mm across the project area is permissible, a Plane grid can be extended 10 km in Eastings either side of the Plane datum origin with an assumption that the scale factor will be 1. The elevation range is more critical as a 1 km line will change in projection length by 8 mm for every 100 metres difference in elevation at each end of the line. This is an important consideration for cadastral surveys and engineering projects with large changes in elevation. For example, a 600 metre line (horizontal component at mean ground elevation) with a difference in elevation of 400 metres, would be 600.019 metres at the upper ground elevation and 599.981 metres at the lower ground elevation. Food for thought maybe (Figure 6).

5   Worked example of a PNG94 cadastral survey
Project: MORO

5.1   Requirement
1. Establish PNG94 cadastral control in the Moro area
2. Establish a Moro Plane Grid to be used for cadastral surveys in the Moro area

5.2   Planning
Two old PSMs (PSM 17742 and PSM 17741) have been located at each end of the Moro Airstrip. The PSMs do not have PNG94 coordinates, but are in ideal locations for GNSS observations.
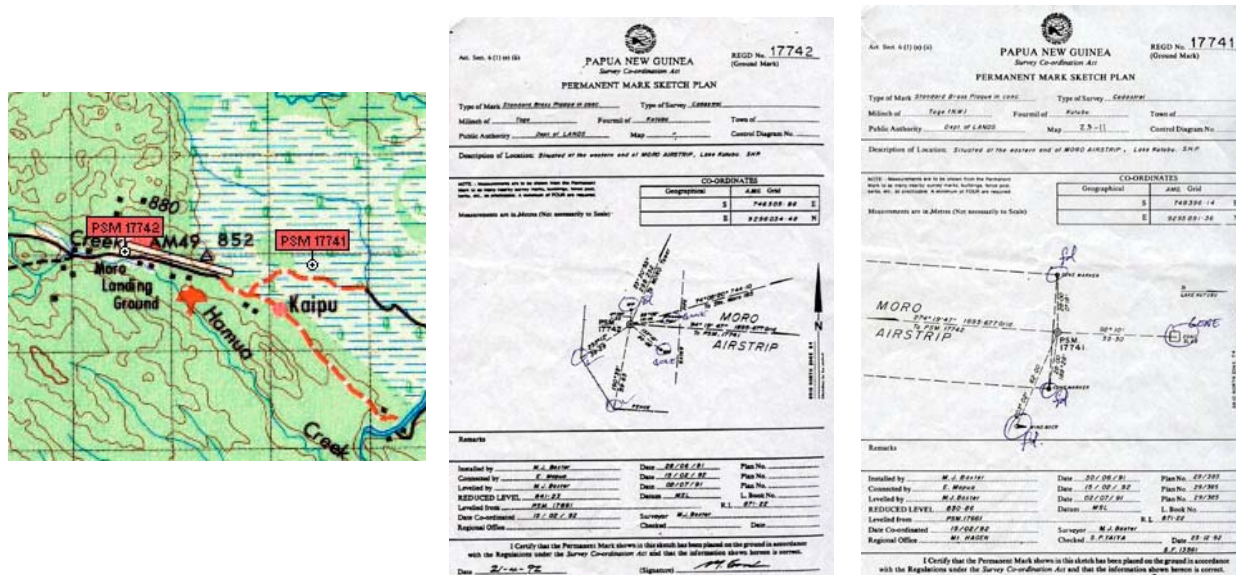

Figure 7. Location diagram and PSM sketches

5.3   Observations
At the time of the survey, there was no fiducial PNG94 control point within 50 km of Moro, so AUSPOS will be used to establish PNG94 in the area.

A dual-frequency receiver is setup on PSM 17742.

5.31 The following receiver checks are done beforehand:
1. The receiver has enough memory (old jobs backed up and deleted)
2. The batteries are charged and voltage tested
3. The elevation mask is set to 10 degrees
4. The epoch interval is set to 30 seconds (AUSPOS/NRCan), or 10 seconds for static surveys
5. The receiver is set to record all observables

5.32 The antenna is setup over the PSM and the following things are checked:
1. Check that the site is clear of trees and obstructions (trim & cut down vegetation)
2. Check that the tripod is stable
3. Check the optical plummet centering with a plumb-bob
4. Check that the antenna is levelled correctly (check that spot bubble is in adjustment)
5. Measure the height to the edge of the antenna at three places around the antenna
6. The measurement should not differ by 1-2 mm. Log the measurement in the site log
7. Sketch the antenna in the site log and identify the point where measurement was made
8. Note down the antenna type and serial number in the log

9. Start the receiver and note the PNG start time and date in the log
10. Check that the receiver has acquired satellites and is logging data
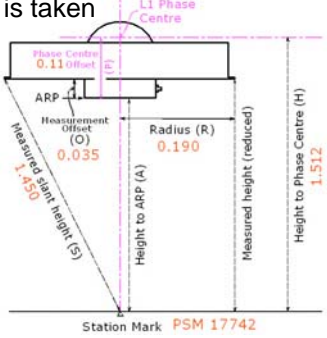
GPS Occupation Log		Antenna sketch
Site ID or filename	*7742*	show point where measurement
Station Name	*PSM 17742*	to antenna is taken
Antenna type	*Ashtech Chokering*	
Antenna serial number	*860*	
Height measurement (start)	*1.450*	
Height measurement (end)	*1.450*	
Height to Phase Centre	*1.512*	
Date start	*5/12/07*	
Time start (PNG Time)	*12:24*	**Approximate position**
Time end (PNG Time)	*3:35*	Latitude *S 6°21'45"*
Date end	*6/12/07*	Longitude *E143°13'46"*

Figure 8. GPS Occupation data log for occupation

5.33   Compute the Height to the phase centre:
Converting slant heights (*S*) to Phase Centre heights (*H*)

The general forumula is: $H = \sqrt{S^2 - R^2} - O + P$

For an Ashtech Chokering (outer edge of chokering) use: $H = \sqrt{S^2 - 0.19^2} - 0.035 + 0.11$

S (Slant height measurement) is 1.450 then *H* = 1.512 using the above equation
Transcribe this phase centre height onto the log sheet

The receiver is run for at least seven hours in order to obtain sufficient observations for AUSPOS

5.34   Shutdown and download
Check the following things before packing up:
1. Check that the bubble is still centered (in case setup was disturbed during observations)
2. Recheck the antenna heights to verify that the antenna has not been disturbed
3. Shutdown the receiver and note the end time and date on the log sheet
4. At the office, download the raw data from the receiver using download software
5. Convert the raw data file to RINEX format using Rinex conversion software
   (The receiver download software often has a RINEX converter included, or use **teqc.exe**)
6. Compute the height to the Antenna Phase Centre if not already done in the field

5.4   Processing with AUSPOS
Two days after the observations, submit the RINEX file to the AUSPOS web-site,
        http://www.ga.gov.au/bin/gps.pl

By waiting 2 days after the observations have been completed, the data will be processed using the IGS Rapid Orbit

        Type in antenna phase centre height measurement (1.512) in the height box

        Leave the antenna type as **"Default"**

Type in your email address, so that you can receive the report.

5.41 Evaluating the AUSPOS solution

Review the AUSPOS processing report that is emailed back.

1. First check that the entered antenna height is correct.

Because you have selected the Default antenna type, the entered height is assumed to refer to the phase centre

1 User and IGS GPS Data

All antenna heights refer to the vertical distance from the Ground Mark to the Antennna Reference Point (ARP). (Ignore this statement, you have already calculated the height to Phase Centre)

User File	Antenna Type	Antenna Height (m)	Start Time	End Time
77423391.070	DEFAULT(NONE)	1.5120	2007-12-05 02:24:00	2007-12-05 17:36:00

File submitted

Phase Centre Height

2. Check that the IGS Rapid or Final orbit is used

(Final Orbit available between 2-3 weeks after observations)

2 Processing Summary

Should be IGS Rapid or Final (Not Ultra-Rapid)

Date	IGS Data	User Data	Orbit Type
2007-12-05	jab1 tow2 darw	7742	IGS Final

3. Check the coordinate precision statistics at the end of the report.

The sigmas should be less than 0.015 for a reliable solution.

5.2 Apriori Coordinate Updates - Cartesian, per day

	dX(m)	dY(m)	dZ(m)	yyyy/mm/dd
7742	-0.028	0.033	-0.002	2007/12/05

5.3 Coordinate Precision - Cartesian, per day

	sX(m)	sY(m)	sZ(m)	yyyy/mm/dd
1 Sigma				
7742	0.009	0.009	0.005	2007/12/05

Should be better than 0.015

5.4 RMS, Observations, Deletions per day

Data	RMS (m)	# Observations	% Obs. Deleted	Date
darw	0.0094	9940	4 %	2007-12-05
tow2	0.0086	9971	3 %	2007-12-05
jab1	0.0102	8995	5 %	2007-12-05
7742	0.0094	28906	4 %	2007-12-05

5.5 Compute the PNG94 coordinates of the site from AUSPOS

1. Extract the ITRF2005 ellipsoidal coordinates of the site from the report.
DO NOT USE THE GDA94 SOLUTION IN PNG!!

4.2 Geodetic, GRS80 Ellipsoid, ITRF2000

The height above the Geoid is computed using the GPS Ellipsoidal height and subtracting a Geoid-Ellipsoid separation. Geoid-Ellipsoidal separations, in this section, are computed using a spherical harmonic synthesis of the global EGM96 geoid. More information on the EGM96 geoid can be found at earth-info.nga.mil/GandG/wgsegm/egm96.html

	Latitude(DMS)	Longitude(DMS)	Ellipsoidal Height(m)	Above-Geoid Height(m)
darw	-12-50 -37.3329	131 7 57.8646	125.142	74.992
tow2	-19-16 -9.4037	147 3 20.4798	88.165	30.118
jab1	-12-39 -31.8491	132 53 38.0363	82.161	27.824
7742	-6-21 -44.8827	143 13 46.1084	917.863	838.288

ITRF2000 / WGS84 Coords of site (Not PNG94 coordinates)

2. Convert these coordinates to PNGMG/UTM coordinates using the spreadsheets described in Section 6, or other Geographical Calculator ensuring that the WGS84 or GRS80 ellipsoid is used.

The ITRF2005 UTM coordinates at the epoch of observation are:

7742   UTM Zone 54   E 746627.938   N 9296195.280

3. Determine what the site velocity is for the location (refer section 3.62)
A recent report has the following ITRF site velocity for the Kutubu area.

Eastings +33 mm/yr Northings +54 mm/yr

Use the spreadsheet ***wgs84itrf_pngmg_convert.xls*** desribed in section 6 to compute PNG94 or:

4. Compute the epoch of measurement

The date of the survey is 5th December 2007, so the day of year is 339. This can be obtained from a Julian Day calendar, or more simply from the Rinex file submitted check the RINEX file submitted ->:

The measurement epoch is 2007 + 339/365
= **2007.929**

The number of years since the PNG94 reference epoch (1st January 1994) is   1994.0 - 2007.929
= **-13.929**

1   User and IGS GPS D

All antenna heights refer to the vertica (ARP).

User File	Antenna Ty
7742 339 1.070	DEFAULT(NONE)

Day of Year

The displacement in Eastings is computed by multiplying the site velocity by the difference in epoch between the reference epoch and epoch of measurement.

This can be computed using the equations *(3.620 and 3.621)*:

$$E_{PNGMG} = E_{UTM(ITRF)} + V_E(1994.0 - Y_M)$$
$$N_{PNGMG} = N_{UTM(ITRF)} + V_N(1994.0 - Y_M)$$

where,

E_{PNGMG} is the PNGMG Easting
N_{PNGMG} is the PNGMG Northing
$E_{UTM(ITRF)}$ is the ITRF UTM Easting derived from the AUSPOS solution
$N_{UTM(ITRF)}$ is the ITRF UTM Northing derived from the AUSPOS solution
V_E is the site velocity Easting component
V_N is the site velocity Northing component
Y_M is the epoch of measurement

Substituting the values into the equations above gives us:

$$E_{PNGMG} = 746627.938 + 0.033(1994.0 - 2007.929)$$
$$N_{PNGMG} = 9296195.280 + 0.054(1994.0 - 2007.929)$$

so the final coordinates are:

PSM 17742   PNGMG94 Zone 54   E 746627.478 N 9296194.528

5.6   Compute the local height datum offset to be applied

AUSPOS has used the EGM96 derived geoid model to compute the MSL elevation of PSM 17742, which is 838.288 metres. Inspection of the existing survey control for the PSM gives an MSL RL of 841.23 metres. Since this value forms the basis of existing level control in the area, a correction should be computed so that other GPS surveys that initially use EGM96 can be adjusted to fit the existing level datum.

The correction to be applied to subsequent GPS surveys is 841.23 - 838.288 = 2.94 metres

In other words EGM96 derived MSL is 2.94 metres less than the existing height datum (which can be called the Moro Height Datum). So 2.94 metres needs to be added to all other EGM96 derived surveys in the Moro area in order to maintain parity.

5.7   Datum verification and local connections by baseline processing

Once the PNG94 origin has been established, survey another datum point

1. Set up the base station at PSM 17742 (repeat the observation procedure described earlier)
2. Set up the rover station at PSM 17741 (repeat the observation procedure described earlier)
Change the epoch interval to 10 seconds for shorter occupation times.

Because the baseline is short, only 20 minutes of observations are required

Download and process the baseline
1. Download the base receiver raw data (and convert to RINEX if necessary)
2. Download the rover receiver raw data (and convert to RINEX of necessary)

3. Start up the baseline processing software
(Trimble Geomatics Office used in example)

4. Start a new project called "MORO"

5. Setup the datum, projection and height system: e.g.

Units: Metric
Proj. System: UTM
Zone: 54 (Southern Hemisphere)
Datum: WGS1984 (for processing is OK)
Geoid Model: Use EGM96

6. Load in the data from each end of the baseline (Import RINEX, or load raw file directly)

Note: Nav files are required

7. Check that the heights are correct, select "antenna phase centre"

8. Hold the base station fixed by editing the coordinates with the PNG94 / PNGMG94 coordinates

Use ellipsoidal height for processing. Leave elevation (MSL) blank at this stage

9. Process the baseline
10. View the baseline processing report. Check to see if the baseline is fixed. Look at the RMS, Variance and Ratio

11. Print off or copy the computed coordinates

PSM 17741
PNGMG94 Zone 54
E 748517.451 N 9296051.435

12. Apply the local height datum correction

Moro Height Datum = EGM96 + 2.94

= 827.428 + 2.94 = **830.37**

5.8   Establish a Moro Plane Datum for cadastral surveys in the Moro area

PSM 17742 is fairly central to the Moro area and the elevation difference is mostly < 100 m in the areas where any construction is likely to occur. The following strategy can be used to establish a Moro Plane Datum using PSM 17742 as a datum origin.

The PNGMG94 coordinates are of the Moro Plane Datum origin are:
PSM 17742 PNGMG94 Zone 54 E 746627.478 N 9296194.528

The equivalent Moro Grid coordinates could be:
PSM 17742 Moro E 46627.478 N 96194.528 (PNGMG E - 700000, PNGMG N - 9200000)
Azimuth same as PNGMG94 at origin

Moro Scale Factor 1.0000000 at PSM 17742 RL 841.23 (Moro Height Datum)
The combined height and PNGMG Grid Scale factor at PSM 17742 is 1.000208 (using ellipsoid height)

Using formulae (4.21 to 4.24):

$$E_{MORO} = E0_{MORO} + \frac{1}{k_p}(E_{PNGMG} - E0_{PNGMG}) \text{ ,or}$$

$$E_{MORO} = 46627.478 + \frac{(E_{PNGMG} - 746627.478)}{1.000208}$$

$$N_{MORO} = N0_{MORO} + \frac{1}{k_p}(N_{PNGMG} - N0_{PNGMG}) \text{ ,or}$$

$$N_{MORO} = 96194.528 + \frac{(N_{PNGMG} - 9296194.528)}{1.000208}$$

$$E_{PNGMG} = E0_{PNGMG} + k_p(E_{MORO} - E0_{MORO}) \text{ , or}$$

$$E_{PNGMG} = 746627.478 + 1.000208(E_{MORO} - 46627.478)$$

$$N_{PNGMG} = N0_{PNGMG} + k_p(N_{MORO} - N0_{MORO}) \text{ ,or}$$

$$N_{PNGMG} = 9296194.528 + 1.000208(N_{MORO} - 96194.528)$$

6 Spreadsheets for geodetic computations

A number of different free spreadsheets have been developed by Quickclose for geodetic computations in PNG. These are available on the web:

PNG94 Ellipsoidal coordinates (latitude & longitude) in decimal degree format can be converted to PNGMG94 (UTM) Coordinates using spreadsheet:
PNG94Ellipsoid_to_PNGMG94.xls or http://www.quickclose.com.au/PNG94Ellipsoid_to_PNGMG94.xls
(Grid convergence, Point Scale Factor and Combined Height and Grid Scale factor also computed)

PNGMG94 (UTM) Coordinates can be converted to PNG94 Ellipsoidal coordinates using spreadsheet:
PNGMG94_to_PNG94Ellipsoid.xls or http://www.quickclose.com.au/PNGMG94_to_PNG94Ellipsoid.xls
Note: The correct PNGMG94 zone must be selected, otherwise longitudes will be incorrect.
(Grid convergence, Point Scale Factor and Combined Height and Grid Scale factor also computed)

To convert angles in DD.MMSSSS format (for example -3°05′09.3″ typed as -3.05093) use:
dms_to_dd.xls or http://www.quickclose.com.au/dms_to_dd.xls

To convert angles in Degrees, Minutes and Seconds format (for example -3°05′09.3″ typed as -3 05 09.3) in separate columns use:
deg_min_sec_to_dd.xls or http://www.quickclose.com.au/deg_min_sec_to_dd.xls

To convert angles in Decimal degrees to Degree, Minute and Seconds format use:
dd_to_dms.xls or http://www.quickclose.com.au/dd_to_dms.xls

To convert Cartesian coordinates to ellipsoidal (latitude, longitude) coordinates use:
cart_to_ell.xls or http://www.quickclose.com.au/cart_to_ell.xls
or **cart_to_dms.xls** or http://www.quickclose.com.au/cart_to_dms.xls
or **cart_to_dd.xls** or http://www.quickclose.com.au/cart_to_dd.xls

To convert ellipsoidal coordinates (in decimal degree format) to Cartesian format use:
dd_to_cart.xls or http://www.quickclose.com.au/dd_to_cart.xls

To convert ITRF2005 and WGS84 UTM coordinates to PNGMG94 Grid coordinates with a known site velocity
wgs84itrf_pngmg_convert.xls or http://www.quickclose.com.au/wgs84itrf_pngmg_convert.xls

Appendix 1 PNG94 1st order control listing - Provisional update 7th June 2008 (verification required)

Station location			PNG94 Ellipsoidal Coordinates			PNGMG94 Grid Coordinates			MSL	Site Velocity	
Location	GPS ID	NMB Reg. No.	Latitude	Longitude	Ellipsoid Height	Zone	Easting	Northing	RL	E m/yr	N m/yr
Aiambak	AIAM	PSM 9550	-7°20'51.8206"	141°16'01.4470"	95.52	54	529475.73	9187801.94	21.7	0.037	0.058
Alotau - Gurney Airport	ALT2	PSM 9538	-10°18'37.5094"	150°20'18.0912"	94.87	56	208478.37	8859053.57	16.3	0.031	0.058
Buka Airport	BUK1	PSM 4871	-5°25'34.3712"	154°40'08.4373"	73.25	56	684918.22	9399967.57	4.3	-0.059	0.031
Daru	DARU	AA 440/A	-9°05'15.5229"	143°12'27.1952"	80.28	54	742639.83	8994719.42	4.9	0.035	0.055
Finschhafen	FINS	PSM 19471	-6°36'55.4209"	147°51'17.6868"	74.24	55	594504.66	9268686.35	9.5	-0.006	0.004
Goroka - Airport	GOKA	PSM 9833	-6°04'53.0717"	145°23'30.4470"	1664.47	55	322023.98	9327531.64	1585.4	0.023	0.046
Hoskins - Airport	HOSK	PSM 9795	-5°28'00.4073"	150°24'31.6614"	101.35	56	212869.72	9395119.32	18.0	0.022	-0.027
Kavieng - Airport	KAVI	PSM 9513	-2°34'53.0660"	150°48'22.5361"	78.81	56	256077.96	9714464.61	2.7	-0.067	0.027
Kenabot - Lands Base	KENB	PSM 23342	-4°20'45.1168"	152°16'07.9951"	136.69	56	418875.65	9519602.79	63.2	-0.002	-0.041
Kerema - Catholic Mission	KERE	PSM 31703	-7°57'28.0191"	145°46'19.0726"	97.57	55	364647.58	9120168.45	21.5	0.030	0.052
Kikori - Airport	KIKO	PSM 5583	-7°25'24.6531"	144°14'55.7677"	88.93	55	196298.45	9178490.00	12.01	0.035	0.054
Kiunga - Airport	KIUN	PSM 9465	-6°07'37.9805"	141°16'41.2696"	103.27	54	530773.45	9322724.61	27.7	0.038	0.056
Lae - Unitech DSLS Base	LAE1	PSM 31107	-6°40'25.3661"	146°59'35.4668"	140.37	55	499246.79	9262320.80	67.12	0.026	0.052
Lae - Unitech Sports	9799	PSM 9799	-6°40'16.9707"	146°59'52.3754"	130.31	55	499765.91	9262578.60	57.06	0.026	0.052
Lake Kopiago - Airport	KOPI	PSM 17001	-5°23'09.0852"	142°29'42.1907"	1412.79	54	665650.98	9404480.51	1327.7	0.031	0.055
Losuia	LOSU	AA 583	-8°32'07.2596"	151°07'30.8181"	85.16	56	293644.60	9056016.40	6.1	0.021	0.071
Madang - Airport	MAD1	GS 15495	-5°12'41.2891"	145°46'56.1940"	73.27	55	365044.17	9423829.87	5.0	0.023	0.039
Manus - Lombrum Secor	MANU	PSM 9522	-2°03'02.2944"	147°21'37.6363"	129.77	55	540084.32	9773337.48	50.8	-0.065	0.027
Mendi	MEND	PSM 3507	-6°08'36.7344"	143°39'22.1658"	1815.08	54	793981.21	9320198.80	1732.6	0.029	0.047
Misima - Airport	MIS1	PSM 9195	-10°41'19.9049"	152°49'58.9388"	87.46	56	481741.61	8818417.91	13.1	0.030	0.055
Moro - Airport	MORA	PSM 17442	-6°21'44.9072"	143°13'46.0940"	917.86	54	746627.49	9296194.53	837.4	0.033	0.054
Mount Hagen - Airport	HGEN	PSM 3419	-5°49'55.7591"	144°18'23.7948"	1710.15	55	201725.79	9354636.51	1626.5	0.030	0.048
Nadzab - Airport	NADZ	ST 31024	-6°33'47.9879"	146°43'39.6541"	148.83	55	469894.96	9274514.88	77.4	0.024	0.056
Namatanai - Airport	NAMA	GS 19461	-3°39'58.5422"	152°26'06.1582"	114.96	56	437261.32	9594742.59	43.9	-0.061	0.001
Nogoli Hides - Helipad	NOGO	PSM 30041	-5°56'02.4348"	142°47'16.7455"	1340.20	54	697930.59	9343770.78	1257.5	0.032	0.054
Pomio	JACQ	PSM 9515	-5°38'42.9782"	151°30'19.6067"	151.55	56	334476.29	9375795.22	77.3	0.020	-0.053
Popondetta	POPN	PSM 9371	-8°46'09.6499"	148°14'00.3966"	187.53	55	635667.54	9030425.34	106.8	0.024	0.054
Port Moresby - NMB Base	MORE	PSM 15832	-9°26'02.7696"	147°11'12.2016"	116.74	55	520498.42	8957148.59	41.3	0.028	0.053
Rabaul - RVO Base	RVO_	RVO	-4°11'27.1915"	152°09'49.5108"	266.24	56	407190.52	9536723.33	191.9	0.007	-0.052
Tokua - Airport	TOKU	GS 9822	-4°20'27.7832"	152°22'45.8215"	82.05	56	431137.64	9520146.01	9.5	-0.010	-0.036
Vanimo - Doppler	VANI	PM 63/1	-2°41'05.2819"	141°18'15.6562"	80.59	54	533829.65	9703242.49	3.4	0.013	0.045
Wankkun - Pillar	NM34	NM/J/34	-6°08'52.0739"	146°04'52.4422"	509.98	55	398344.12	9320370.15	436.7	0.026	0.047
Wau - MCG Base New	WAU1	WAU1	-7°20'57.0996"	146°42'55.7613"	1224.79	55	468599.31	9187638.65	1144.5	0.025	0.056
Wewak - Airport	WEWK	PSM 15497	-3°35'02.5848"	143°40'00.1481"	83.91	54	796268.18	9603418.22	5.8	0.017	0.053
Wuvulu	WUVU	PSM 15456	-1°44'07.5951"	142°50'10.0781"	79.03	54	704257.66	9808081.66	2.4	-0.068	0.019

Horizontal Coordinates - Positional Uncertainty < 0.05m, Ellipsoidal Heights - Uncertainty < 0.10m, MSL RLs - Uncertainty < 0.5m (except Lae & Kikori < 0.10m)

* Coordinates require verification by resurvey

Workshop: **Connecting a Cadastral Survey to PNG94 using GNSS**