Establishment of a GNSS Testing and Validation Facility in Perth, Western Australia

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ABSTRACT

A facility for testing and validating GNSS (Global Navigation Satellite Systems) hardware, software/firmware and operators has been established on and close to the Bentley Campus of Curtin University of Technology. This comprises a network of fixed pillars that can be used for testing static-based GNSS positioning and a network of ground marks that can be used for testing real-time stop-and-go kinematic (RTK) GNSS positioning. This paper summarises the establishment and coordination of these networks and presents a case study of the validation of three RTK GPS systems (Trimble, Ashtech and Leica) and their operators for height determination on the Australian Height Datum, which was undertaken in order to set highway surveying standards in Western Australia. It is demonstrated that RTK GPS systems remain subject to errors that are not reported to the operator, such as incorrect integer ambiguity resolution. Further work, in areas such as algorithm development and/or improved procedures, is thus required to make RTK GPS a more robust and reliable positioning tool. Until this is achieved, users must remain sceptical about the positional accuracy reported by such systems and apply additional, independent quality control and quality assurance procedures, such as the use of this facility.

1. INTRODUCTION

Global Navigation Satellite Systems (GNSS) are being used successfully for many surveying and navigation applications around the world. These applications range from high-precision geodetic datum definition to the near-instantaneous positioning and navigation of moving objects. However, the use of GNSS does not differ from conventional surveying techniques in that quality assurance processes must be utilised on a routine basis. This is essential to ensure that satisfactory accuracy specifications can be, and are being, met.

There has been little independent, objective testing of the precision, accuracy and reliability of GNSS systems and positioning procedures reported in the geodetic-surveying literature (eg. Wylde and Featherstone, 1995; Edwards *et al*., 1999; Stewart *et al*., 1998; Featherstone and Stewart, 2001). Instead, the large majority of users of commercial GNSS hardware and software/firmware (herein collectively termed the GNSS systems) appear to rely largely upon the internal precision estimates provided by the software/firmware. In some cases, users even seem to rely upon the manufacturers' brochures, which tend to claim performance based upon operation in somewhat ideal conditions.

As an example, the short station occupation times used for real-time kinematic (RTK) GPS positioning (i.e. stop-and-go) are influenced greatly by the effects of multipath and incorrect integer ambiguity resolution that are not necessarily reported to the user. Therefore, the need for external, independent, objective testing and validation of GNSS systems in realistic operating conditions becomes obvious. Coupled with this, is the need for proper operational practice and procedures to be adhered to routinely when using GNSS in a production mode. However, the level of expertise held by the users of GNSS varies widely, with many having little appreciation of the numerous factors that act to degrade the accuracy of the determined coordinates.

This paper describes the establishment of a testing and validation facility for GNSS systems and their operators on and close to the Bentley Campus of Curtin University of Technology, Western Australia. A case study, excerpted from Featherstone and Stewart (2001), is presented which illustrates the use of the facility for a combined validation of RTK GPS systems (ie. hardware and software/firmware) and their operators for ellipsoidal height determination and subsequent transformation to the Australian Height Datum (AHD). Since there is little or no control over the data collection, reduction and coordinate transformation strategies adopted, this case study only allows for a *combined* validation of the RTK GPS systems and their operators.

2. THE NEED FOR QUALITY CONTROL IN GNSS POSITIONING

The current status of GNSS technology, in terms of algorithms, software/firmware, hardware and operational practice and procedures, is such that internal quality control indicators of coordinate precision, provided by proprietary software/firmware, *cannot* be used solely to validate GNSS-based positioning accuracy. Moreover, the site- and time-dependent nature of most GNSS errors precludes the assumption that if a GNSS-derived position is accurate at a single point, it must be accurate for all other points. This problem becomes exaggerated for stop-and-go and pure kinematic GNSS applications, where these errors tend to manifest as positional biases over the short occupation periods used.

To properly estimate the accuracy of the coordinates computed by a GNSS system, redundancy in the form of additional, independent observations is required. One useful field practice is to make check measurements during each and every GNSS survey at multiple points interspersed throughout the area. However, this approach - though commendable - is by no means ideal. It does not guarantee that the same level of accuracy will be achieved for all other points in the survey. Furthermore, the problem with using only one independent check measurement at each point in the survey is that it is impossible to discern between correct and incorrect points at definite levels of accuracy, precision and significance. Accordingly, the burden of proof remains with the user to ensure that the positions are accurate to the desired standards at all times and places within the GNSS survey.

Probably the only really rigorous approach to independently estimate the uncertainty in GNSS-derived positions is to make several separate observations at each and every point. This could use the GNSS data received from other or multiple base-stations, and/or observed at different sidereal times to ensure that the satellite geometry and multipath effects have changed sufficiently. Alternatively, *in situ* check measurements at each and every point could use independent positioning technologies, such as inertial navigation systems, that are of a similar or better accuracy.

It is essential to reiterate that the accuracy of any GNSS-derived position is spatially and temporarily dependent. There is simply no guarantee that just because a GNSS system delivers results of a particular standard at a particular place and time that it will in others. In the absence of independent positioning as a check at all points, either post mission or *in situ*, one interim solution is to validate the GNSS system (and its operator) over a test facility that hosts conditions similar to those experienced in practice. This type of validation should be performed both before and after the GNSS survey or, at the very least, when the software/firmware is updated and/or the hardware is serviced or repaired. The inference is that the same GNSS system operated in the same way (by the same operator) is likely to perform to a similar standard elsewhere, given a similar operating environment. Importantly, however, this is only ever an inference and never offers any guarantee of positional accuracy.

However, the above considerations of GNSS-derived positional accuracy and reliability have to be balanced against practical and cost considerations. The most rigorous approach of using validation by multiple, independent measurements, or complementary positioning systems *in situ*, at each and every point is a more laborious and expensive approach. Therefore, the use of a test facility to validate both the GNSS system and its operation (by the same user(s) that will conduct subsequent GNSS surveys) becomes a considerably more attractive option.

3. ESTABLISHMENT OF THE VALIDATION FACILITY

The GNSS validation and test facility established in Perth, Western Australia, comprises a well-controlled geodetic network of permanent monuments (fixed pillars and ground marks), whose coordinates are known in three dimensions on the Geocentric Datum of Australia 1994 (GDA94) and on the Australian Height Datum (AHD), and associated computer software that analyses the results taking into account the uncertainty on the control coordinates.

The GNSS system operator first occupies these points, then the GNSS-derived three-dimensional coordinates are compared with the ‘known’ coordinates using the above computer software, assuming that the most accurate geodetic datum transformations have been applied where appropriate. To introduce an additional element of independence, accurate coordinates of all the control points are not supplied to the users of the facility. The users are also requested to conduct the GNSS surveys in a ‘production mode’ so as to faithfully replicate the procedures that are likely to be used in practice.

High-precision terrestrial- and GPS-geodetic surveys were conducted in 1997 to provide the control coordinates of the monuments through a collaborative venture among the Geodesy Group, Department of Spatial Sciences, Curtin University of Technology, the Western Australian Department of Land Administration (DOLA) and Main Roads Western Australia (MRWA). Importantly, part of the facility is connected geodetically to an existing EDM calibration baseline, which is accepted as part of the national length standard (Commonwealth of Australia, 1960). This allows for a certain (but as yet untested in the law courts) level of ‘legal traceability’ to the GNSS systems validated on the facility.

The monuments comprising part of the facility were installed in two phases, and each provides slightly different validation of GNSS systems, as follows.

- One part of the facility uses permanent, forced-centring pillars that are connected geodetically to an existing EDM calibration baseline. This part of the facility has been used previously in an attempt to provide some legal traceability to the use of GPS and GLONASS systems (Stewart *et al*., 1998).
- The other part of the facility uses a combination of standard survey marks of the Western Australian geodetic network, some of the above-mentioned fixed pillars, and ground marks on and around the Bentley Campus. This part of the facility has been used previously to set standards for RTK GPS surveys of roads for MRWA (Featherstone and Stewart, 2001).

  Since the ground mark component of the facility does not allow for exact centring of the GNSS antennas, there is inevitably an element of the validation that includes the ability of the user to place the antennas accurately and vertically over the ground marks. It is argued here that this represents a more realistic replication of the way in which stop-and-go RTK GNSS positioning is applied in practice.

3.1 The Fixed-pillar Component

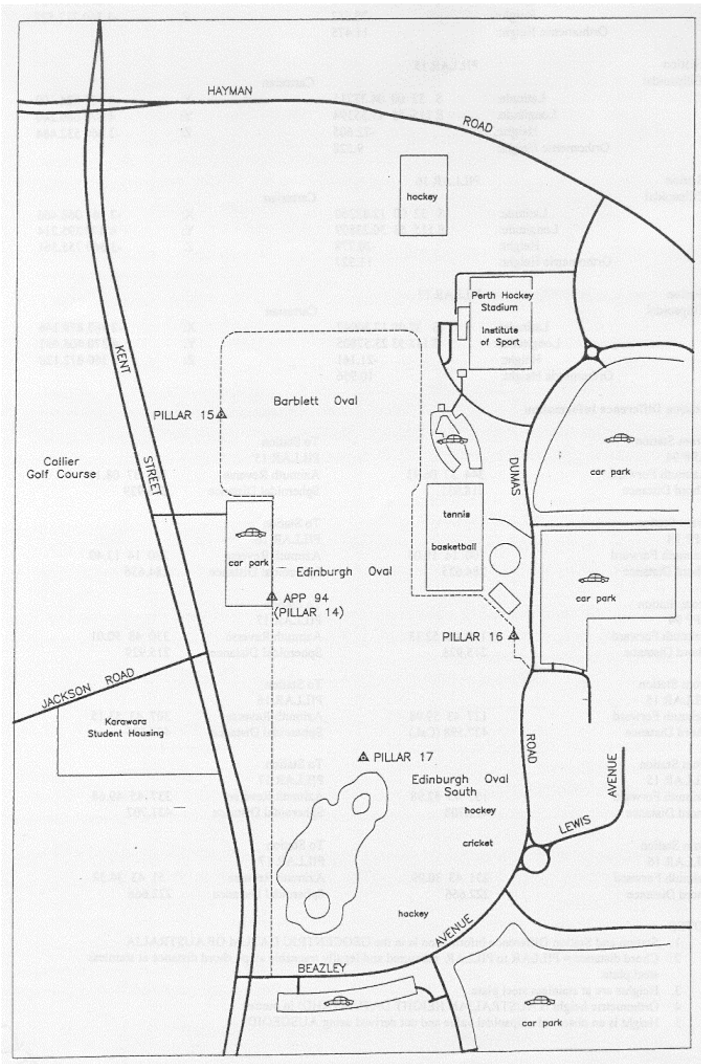

Figure 1. The fixed-pillar component of the GNSS validation facility (courtesy DOLA); see Figure 2 for a locality map.

In 1992, an EDM calibration baseline was established at the north-western end of Curtin University's Bentley Campus through a collaborative venture between the then School of Surveying and Land Information (now the Department of Spatial Sciences), Curtin University of Technology and DOLA. This EDM baseline comprises part of the national standard for length (Commonwealth of Australia, 1960). The EDM pillars and GPS pillars (described next) are regularly re-observed using an EDM that has a current Regulation 80 certification from the National Measurement Laboratory. The instrument used is normally a *Mekometer* hired from the Tasmanian Hydro-electric Authority.

Substantial obstruction of the sky by foliage renders most fixed pillars of the existing EDM calibration baseline unsuitable for GNSS tests. Therefore, four additional fixed pillars (APP-94, Pillar 15, Pillar 16 and Pillar 17) were established on the periphery of nearby University sports fields (Figure 1). The network geometry of the fixed-pillar component of the facility was constrained by the existing physical environment, such as buildings and underground power lines. The additional pillars are inter-visible by terrestrial geodetic survey, with the exception of Pillar 15 to Pillar 16, and have a generally unobstructed view of the sky above a 15° elevation. All fixed pillars are set in secure concrete foundations and have a machined stainless steel top plate and 5/8" thread to allow forced centring of the GNSS antennas.

The additional fixed-pillars were geodetically surveyed with respect to the existing EDM calibration baseline using Australian Class A terrestrial survey techniques (ICSM, 1996) and also tied via GPS to high order (*ibid*.) control of the Western Australian geodetic network. The semi-major axes of the relative error ellipses resulting from a constrained least squares adjustment range between 3mm and 6mm (95% confidence). The relative errors, expressed in millimetres per kilometre of inter-station distance, range between 9ppm and 80ppm (95% confidence), with larger values generally being for shorter baseline lengths (Stewart *et al*., 1998).

The ellipsoidal heights of the fixed pillars have been estimated using GPS tied to ITRF-based control coordinates that have 'estimated', rather than 'computed', ellipsoidal heights. These GPS-estimated ellipsoidal heights will be termed 'pure' ellipsoidal heights, because no geoid model has been used to transform optically levelled heights to give 'computed' ellipsoidal heights. Pure ellipsoidal heights are thus insensitive to errors in the AHD and regional geoid model, which can adversely affect computed ellipsoidal heights. Pure ellipsoidal heights are also better suited to testing and validating GNSS systems, because the intrinsic weaknesses due to spirit levelling and regional geoid model errors are avoided.

3.2 The Ground-mark Component

The ground-mark component of the facility was established specifically to set standards for contract surveys conducted for MRWA (Featherstone and Stewart, 2001). It is divided among three sub-networks, each comprising 20 points, which are sited near-linearly on and close to the Bentley Campus (Figure 2). The spatial distribution of these ground-marks was originally designed to make the GNSS tests replicate, as closely as possible, typical highway surveys, with varying degrees of satellite visibility (eg. trees) and site-dependent environmental factors (eg. multipath). Since establishment of this part of the facility, five points along Hayman Road (Figure 2) have, ironically, been destroyed by road works.

A further five existing geodetic control points of the Western Australian geodetic network were included as the RTK GNSS base-stations. These are situated close to the centre of the facility (BASE0) and approximately 1km (BASE1), 2km (BASE2), 5km (BASE5) and 10km (BASE10) from the facility (Figure 2). This allows analyses to be conducted as a function of baseline length from each base-station (i.e. single baseline solutions are used, rather than multi-base-station solutions). These base-stations were deliberately chosen to run in an approximate east-west direction, subject to the placement of existing geodetic control, because the geoid-geocentric-ellipsoid gradient is extremely steep in this direction over the Perth region (cf. Featherstone, 2000). As such, this provides a particularly challenging vertical coordinate transformation from the ‘estimated’ ellipsoidal heights to AHD heights. If the local geoid modelling techniques utilised by the users apply here, it is reasonable to assume that they will apply in other regions where the geoid-geocentric-ellipsoid gradient is not so steep.

To establish this ground-mark component of the facility, a geodetic network was first observed by DOLA using a combination of terrestrial- and GPS-geodetic methods. This network encompasses most existing geodetic control on and around the Bentley Campus, the five RTK base-stations (Figure 2), and three points from the ground-mark network. MRWA subsequently observed the 60 ground-marks within the framework of the DOLA network using fast-static GPS, spirit levelling and terrestrial horizontal geodetic survey measurements. The GPS results have subsequently been checked against independent solutions from terrestrial observations (Thomson, 2000). The combination of GPS- and terrestrial-geodetic methods has allowed coordination of the 60 ground-marks with three-dimensional GDA94 geodetic coordinates and AHD heights.

There is some ambiguity regarding the ‘pureness’ of the GDA94 ellipsoidal heights of these 60 ground marks. This is because, even though they were estimated using GPS, some of the geodetic control points used had their ellipsoidal heights computed from spirit levelled AHD heights and an Australian regional geoid model, whereas some have so-called ‘pure’ ellipsoidal heights. Indirect analysis based on constrained least squares adjustments did not indicate any significant straining of the network adjustment by this mixture of heights. However, future refinements of the facility will introduce ‘pure’ ellipsoidal heights for all remaining ground marks that have not been destroyed, as well as establishing new monuments that will have a better chance of longevity.

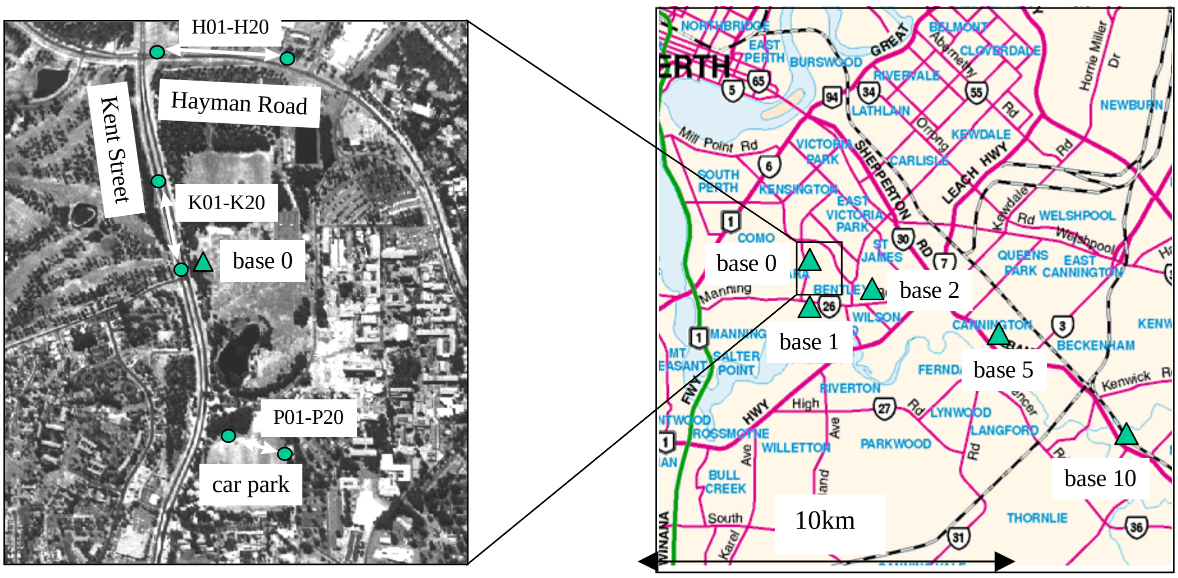

Figure 2. The ground-mark component of the GNSS validation facility along Hayman Rd, Kent St and Car Park (left) and the five base-stations (right). Maps courtesy of DOLA.

The AHD heights of the 60 ground marks were established with respect to an existing third-order (ICSM, 1996) AHD benchmark. The relative heights were established using Australian Class A levelling techniques (*ibid.*) with a *Leica NA3003* digital level and calibrated invar barcoded staves. Accuracy estimates for geodetic levelling are notoriously difficult to determine from a prescribed survey tolerance for differential spirit levelling. For instance, a field check of misclose will always be subject to the uncertainty of two equal and opposite errors being committed. However, using the Class A allowable misclose (*ibid.*) over the mean distance of 50m between test points *coarsely* implies an estimate of precision of 2mm (95% confidence) in the AHD height at each point.

3.3 The Analysis Software Component

A computer program was written specifically to allow the users of the facility to perform their own verifications of GNSS systems over the ground-mark component. This was conducted as part of contract research from MRWA to validate its RTK GPS survey contractors (Featherstone and Stewart, 2000). The provision of free software adds flexibility to the use of the facility by avoiding the need for the users to engage others to perform the analysis. At the same time, it adds an element of independence by avoiding the supply of accurate control coordinates to the users, thus reducing the temptation to pre-analyse data and reject outliers so as to provide better than actually achieved results.

The software is supplied as an executable file in which the coordinates of the control points are ‘hard wired’ and thus not readily accessible by the user. The user simply inputs their GNSS-derived coordinates of the ground marks into the software, then the software reports whether the results agree with the control coordinates and at what level of confidence to which they agree. Importantly, this analysis takes into account the uncertainty in the control coordinates. The results of the tests are supplied to MRWA as part of any tender bid that uses RTK GPS in order to demonstrate that the contractor is capable of meeting MRWA’s accuracy specifications for highway surveys. Future work will adapt this software for all points comprising the network to broaden its application and scope.

4. CASE STUDY: ANALYSES OF RTK GPS-DERIVED HEIGHTS

This section summarises some of the pertinent results presented in Featherstone and Stewart (2001) on the *combined* validation of stop-and-go RTK GPS systems and their users. Three different RTK GPS systems (*Ashtech*, *Trimble* and *Leica*), used by three different operators (unnamed), were tested for their ability to determine heights on the AHD using the ground-mark component of the facility. Each user occupied the 60 ground marks relative to each of the five base stations (Figure 2) and the resulting AHD heights were analysed by the Geodesy Group at Curtin University of Technology. As stated earlier, users were instructed to conduct the surveys in a ‘production mode’ so as to replicate the practice and procedures used for actual surveys.

4.1 AHD Height Determination

As alluded to earlier, this case study of AHD height determination presents the most challenging test of RTK GPS systems, principally because height is the least precisely coordinate that is determined by this technique. Coupled with this is the proper transformation of the ellipsoidal heights to the local vertical datum; here the AHD. As such, this case study presents a *combined* test of the RTK GNSS system, the users’ real-time stop-and-go GPS field practices, and their ability to apply local geoid modelling techniques that yield heights compatible with local benchmarks (cf. Featherstone *et al*., 1998).

First, it is important to acknowledge the many assumptions that have had to be used in the analysis. Specifically, the sample sizes per contractor and per base-station are small (≤60 points) and the height differences are not always normally distributed. Also, temporal and spatial correlations exist between adjacent points that are observed a few minutes apart, which is typically the case for RTK GPS surveys. Therefore, all contractors' data were grouped to increase the sample size and thus make inferences based on descriptive statistics more meaningful. Finally, not all contractors provided positions for all 60 points, these being termed 'null solutions', which may bias the conclusion reached.

An additional consideration is that GPS does not measure the height difference between adjacent points, as is the case with differential spirit levelling. Rather, RTK GPS measures the height difference between each point and the base-station. Applying the Australian Class A tolerance (ICSM, 1996) over the maximum baseline length of 10km, and acknowledging the above-mentioned limitations, *coarsely* implies an error of approximately 12mm (95% confidence). Clearly, a compromise needs to be reached, if the performance of GPS is to be assessed as an alternative to differential spirit levelling. Theoretically, the errors attributed to the control AHD heights should be applied over the GPS baseline lengths used during tests of the RTK GPS performance. However, the earlier *coarse* estimate of 2mm (95% confidence) yields a realistic, if not completely rigorous, estimate of the error for AHD control heights in this component of the facility.

Table 1 shows these statistics and the number of solutions grouped by base-station for all contractors' data (excluding BASE10, where there were too many null solutions). From Table 1, the distribution of AHD height differences for all solutions has a mean of −5mm and a standard deviation of ±27mm, which has been calculated from 707 of 724 possible position solutions (i.e. 18 null solutions). This suggests that, *for this survey area and at the time the surveys were conducted* (1997), the stop-and-go RTK GPS-derived AHD heights do not differ statistically significantly from the control AHD heights.

Base-station	*Number of solutions*	*Mean*	*STD*
BASE0	176 of 180	-0.012	±0.024
BASE1	178 of 180	-0.003	±0.027
BASE2	175 of 180	-0.004	±0.029
BASE5	173 of 180	-0.001	±0.028
all	702 of 720	-0.005	±0.027

Table 1. Mean, standard deviation and proportion of the differences in GPS-derived AHD heights for all contractors' solutions for different base-stations (units in metres)

From Table 1, confidence limits can be derived, giving the *expected* range of errors for stop-and-go RTK GPS-derived AHD heights over baselines of up to 5km in length *in areas that exhibit conditions similar to the facility*. Here, a single RTK GPS-derived AHD falls within 53mm of the control value 95% of the time. However, this estimate assumes no errors in the control AHD heights. Propagation of the 2mm (95% confidence) error, estimated for the control AHD heights (see earlier), into the above confidence limit (assuming independence), also gives a 95% confidence of 53mm. This is because the error in the control AHD heights is insignificant in relation to the variability of the GPS-derived AHD height differences. importantly, this confidence limit does not take into account the fact that only 97.7% of the available points had positions supplied by the contractors.

4.2 An Example of Incorrect Ambiguity Resolution

Table 2 shows the effect of a poor position dilution of precision (PDoP) on selected solutions from one user for the case of the BASE5 dataset. This point was observed after points P18 and P19 had been observed with a quite high PDoP and only four GPS satellites. The sequential increase in ellipsoidal height difference from the control values for points P18 to P19 to P20 is indicative of an incorrect ambiguity resolution by the software/firmware.

Test station	*Base-station*	*Number of satellites*	*PDoP*	*Ellipsoidal height difference (m)*
P17	BASE5	4	5.1	0.016
P18	BASE5	4	14.4	-0.181
P19	BASE5	4	15.3	-0.493
P20	BASE5	4	3.42	-0.892

Table 2. Ellipsoidal height differences as a function of varying number of satellites and PDoP. This indicates that an incorrect RTK GPS ambiguity re-initialisation has occurred.

Although a low PDoP is observed at P20 (Table 2), the ellipsoidal height difference is larger than could be expected. This indicates that the RTK GPS software/firmware has fixed the wrong set of ambiguities at an earlier point(s), probably due to the high PDoP and low number of satellites. This also assumes that a re-initialisation occurred before observing point P18, though the contractor recorded no information to be able to verify this. Nevertheless, given the field conditions in this part of the facility, this offers the most plausible explanation.

This type of problem can largely be eradicated using good survey practice. Generally, no RTK GPS solution should be accepted with a PDoP greater than, typically, 6.0 or with less than five visible satellites (eg. Featherstone and Stewart, 2001). Also, collecting continuous RTK GPS data for, say, around ten minutes after re-initialisation *may* have allowed the RTK GPS software/firmware's internal statistical tests to identify the incorrect ambiguity resolution.

Most importantly, this error was not reported to the user, apart from a previously high PDoP and low number of satellites. It was only detected through *in situ* check measurements. The operator of this particular RTK GPS instrumentation has since been able to revise the software/firmware configuration. This example alone clearly demonstrates the usefulness of the facility to validate GNSS systems and their users.

5. DISCUSSION AND RECOMMENDATIONS

This paper has described the establishment of a GNSS testing and validation facility in Perth, Western Australia, and given a case study application of its use in validating the determination of AHD heights using stop-and-go RTK GPS surveys. This has highlighted one case where a RTK GPS system has delivered poor results, apparently unbeknown to the operator. Admittedly, this was probably due to poor survey practice, but exemplifies the need for such validation facilities, if only to test the operator's ability. In a competitive marketplace, it is all too tempting to 'cut corners' at the expense of accuracy, precision and reliability. However, there are the financial and penal risks of operating this way in an increasingly litigious society.

The facility was established with the initial aims of applying some level of legal traceability to GNSS systems (Stewart *et al*., 1998), validating MRWA contractors' use of stop-and-go RTK GPS for AHD height determination (Featherstone and Stewart, 2001) and to set standards and best practices for RTK GPS surveys (Featherstone and Stewart, 2000). However, it can also be used for the validation of other GNSS systems, such as wide-area differential GPS (WADGPS), where the base-stations are situated at a large distance from the facility so as to replicate their practical application. Subject to funding, future work will be conducted to upgrade the facilities to provide improved control coordinates on the GDA94 and more sophisticated analysis software that is easier to use (eg. an on-line facility).

Despite the virtues of using such a validation facility, it is ultimately recommended that multiple *in situ* checks be used at each and every point in a GNSS survey. Where this is not possible, a preferable interim solution is to validate the GNSS system and its operators on such a well-controlled test facility. While not being able to guarantee the results achieved in other survey areas, it at the very least checks that the GNSS system is being operated properly.

Acknowledgments

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